ASSESSMENT OF THE UPGRADING OF THE MAFRAQ WASTEWATER TREATMENT PLANT

September 14, 2005

Prepared for

Executive Summary

Background

The City of Mafraq, located in northern Jordan near the border with Syria, is served by the Mafraq Wastewater Treatment Plant (WWTP). The WWTP facility is situated approximately 6 km north of the city on a 37 hectare (ha) site. The plant was designed for an average daily flow of 1,850 cubic meters, and employs a series of 12 waste stabilization ponds to treat the wastewater generated by the town. The treated effluent from the plant is conveyed to nearby fields and used for agricultural irrigation. Ever since the facility was commissioned in 1988, the plant has experienced problems complying with successive editions of the Jordanian Technical Regulation JS 893 for reclaimed domestic wastewater, including the most recent edition dated 2002 (reference 23).

The objective of this assessment is to assist the Water Authority of Jordan (WAJ) and USAID in identifying a lower technology, low maintenance solution that can be implemented to rehabilitate and expand the Mafraq WWTP.

Approach

In order to assess the performance problems at the plant, we used an approach called the Comprehensive Performance Evaluation/Composite Correction Program (CPE/CCP). The approach was developed by the U.S. Environmental Protection Agency (EPA) in the late-1980s.

The steps in our evaluation were as follows:

- Gather data about the plant from existing reports and studies.
- Visit the WWTP facilities.

- Meet with USAID and WAJ and validate initial impressions.
- Develop and execute a wastewater characterization program.
- Evaluate major unit processes.
- Evaluate the characterization program results.
- Identify performance-limiting factors.
- Screen the rehabilitation alternatives.
- Recommend the most appropriate rehabilitation/upgrading alternative.
- Propose an action plan to design, build and implement the identified improvements to the plant as soon as possible.

Existing Facility

The existing wastewater treatment plant includes a mechanical screen, bypass manual bar rack, and a Parshall flume for influent flow metering followed by two identical biological treatment trains operated in parallel. Each treatment train includes one anaerobic lagoon, three facultative lagoons, and two maturation ponds, configured to be operated in series. Chlorine disinfection of the treated effluent was implemented as part of the initial design however it is not currently in operation. An overall view of the plant is provided in Figure ES-1.

Plant Inspection

The wastewater treatment plant was first visited by Stearns & Wheler staff on May 21, 2005. During the inspection, we interviewed plant staff, observed the operating conditions and measured the surface dimensions of the process units. The measured dimensions were subsequently compared with the dimensions given in the "as built drawings" provided by the Water Authority of Jordan.

Insert Figure ES-1

The measured dimensions are consistent with the dimensions of the ponds found on the "as built drawings". Those dimensions were then compared with values found in three additional sets of documents:

- Mafraq WWTP Monthly Reports to WAJ
- CH2M HILL Reports, including the 2001 Conceptual Design Report
- The RFP issued for the assessment of the rehabilitation potential of the plant

Each of these sources note that "Mafraq WWTP is a Waste Stabilization Pond System with twelve (12) one-half hectare (0.5 hectares) surface area ponds each". This is not the case; they are considerably smaller than this, with individual pond sizes ranging from 0.29 to 0.37 hectare.

Retention time is critical to pond performance because many of the pond's stabilization processes require contact time between the biomass and the many biological and chemical stabilizing influences that exist in a pond

The smaller surface areas noted above also results in smaller volumes of each treatment unit. Hydraulic retention time, which depends on volume (HRT = V/Q, where V is volume and Q is discharge) decreases when the volume of a pond decreases, resulting in less efficient treatment.

Reduced surface areas and volumes of each treatment unit have significantly affected the treatment system since the beginning of operations in 1988. Accumulated biomass and sludge in each treatment stage have also prevented the treatment units from functioning properly.

The maturation ponds were in better shape than the preceding aerobic and facultative ponds. However, they showed signs of sporadic organic overloading as indicated by the blackish traces of dried sludge on their banks.

We noted that influent feed piping into each treatment cell, is located in the center of the short side of each pond and the discharge location is on the opposite sides of the ponds.

This arrangement of the influent and discharge piping is not recommended in current waste stabilization pond (WSP) design manuals. The recommend locations for inlet and discharge pipes are in opposite corners of the ponds. The current arrangement may be promoting short circuiting, further decreasing the hydraulic retention time in each pond. A tracer study could be used to verify this hypothesis

The influent mechanical screen, chlorine contact chamber, Parshall flume, and many of the distribution valves were observed, or reported by the plant personnel to be, not functioning.

Plant operating personnel indicated that flow distribution is not adequate, leading to uneven partition of the flow between the two parallel trains. Plant staff also communicated serious concerns about sludge management practices at the plant, and expressed the desire to have more flexible systems and easier sludge removal options. The last sludge removal from both anaerobic ponds was done 9 years ago. The accumulated sludge volume (since 1996) in both ponds, before dredging and decanting, was calculated at about 6,000 cubic meters which equates to an approximate mass of 500,000 kg using the solids content of about 10 percent.

There are limited, or no, maintenance facilities or laboratory facilities on the site.

Wastewater Characterization Program

A comprehensive wastewater characterization program was planned and executed in May 2005. Sampling (grab and composite) of five wastewater streams was carried out by the Royal Scientific Society (RSS). The samples were collected on the influent wastewater, as well as the effluent side of the anaerobic ponds, the facultative ponds, the maturation ponds and the final effluent. Sludge samples were taken from each series of anaerobic, facultative and maturation ponds. The initial sludge sampling protocol was modified due to difficulties to sample from the center of each basin. Instead, all sludge samples were taken from the berms of each pond.

The characterization program revealed that close to 80 percent of the influent wastewater BOD could be in particulate form and only 10 percent in the readily biodegradable soluble form. During one of the sampling event the measured total BOD was 559 mg/L, of which the soluble (filtered) organic content was 106 mg/L.. During the second characterization event, the total BOD was 567 mg/L and the soluble part 263 mg/L.

The volatile fraction of the suspended solids concentrations in the influent wastewater, observed during the characterization program, was rather significant (66 to 88 percent) . It may be explained by the high retention time in the gravity sewer between Mafraq and the plant. A 6 km sewer line of 900 mm diameter, flowing half full, provides a volume equivalent to about 23 hours retention time at the current average flow of 2,000 m^3/day. The sewer line may be acting as a hybrid anaerobic reactor, degrading a significant portion of the influent BOD_5. The high concentrations of volatile solids in the influent are in fact probably biological matter (bacterial cells), produced during the degradation of the BOD_5.

Sludge concentrations in the basins suggest that volatile solids are drifting from cell to cell. The average total suspended solids (TSS) concentration of the settled sludge at the bottom of the anaerobic ponds is about nine percent (i.e., about 91 percent water content), in the facultative ponds about eight percent (80,000 mg/l), and close to 10 percent in the maturation ponds.

Benthic release occurs when suspended solids settle and dead microorganisms accumulate and a solids layer builds up on the bottom of the lagoons. This layer is decomposed by anaerobic and facultative organisms over time. This process releases organic acids, increasing the BOD in the supernatant. Evaluation of the wastewater characterization data suggests that in addition to the ponds being overloaded due to inadequate surface areas and volumes, their treatment capacity is being further degraded by benthic release of BOD.

A microbiological analysis of the ponds was also conducted. The objective of the microbiological analysis was to diagnose the health of the process and determine the most probable causes for performance problems.

Algae, an indicator of healthy WSPs was absent in the first stages of the facultative ponds. This is a clear indicator of overloaded facultative ponds.

Summary of Performance Limiting Factors

As a result of our plant inspections, wastewater characterization program, microbiological evaluations and discussions with operating staff, we have noted the following ten performance limiting factors, in approximate order of importance.

1. All cells of the plant are smaller than the sizes cited in several key plant documents. Under the current process flow sheet arrangement, the facultative ponds may be undersized by a factor of 10.
2. The first facultative ponds of the existing WSP (F11 and F21) are organically overloaded. In addition, the existing system does not have flexible interconnecting piping, allowing operation of the ponds in parallel or in series. Operation in series of undersized facultative ponds aggravates further the overloading conditions of the system.
3. The long gravity sewer from Mafraq to the WWTP appears to be functioning as an anaerobic reactor, contributing large volume of volatile suspended solids to the plant.
4. Influent and effluent feed lines to each pond are located in the middle of the short sides of the facultative cells, with suspected short-circuiting.

5. Flow splitting between the ponds is unequal between train 1 and train 2, leading potentially to even higher overloading of one of the trains.
6. High benthic feedback of organics from the settled sludge in the anaerobic ponds into the supernatant to the first facultative cells. The anaerobic ponds are operating in fact as anaerobic sludge digesters.
7. Lack of efficient sludge management facilities; solids dredging/pumping equipment and sludge drying system such as sludge drying beds.
8. Lack of laboratory testing facilities on the site.
9. Lack of maintenance facilities on the site, leading to equipment deterioration.
10. Poor flow measurement.

In order for this plant to meet its performance objectives, these performance limiting factors must all be resolved.

Basis of Design for Upgraded Plant

A Design Report was prepared by another engineering firm in 2001 to lay the foundation for upgrades to the plant. This report developed projections for influent flows and loads that we have used as a basis for our evaluation. We relied on the projections in this report and did not develop any flow projections or estimated pollutant loadings independently. We believe that one characterization program of 2 days, cannot replace the analysis of the local knowledge of wastewater characteristics, flows, loads and future population. In addition USAID provided very clear instructions to use the data from the 2001 study.

Key flow projections for the Conceptual Design are as follows:

2025 Average Daily Flow	6,550 m^3/day
2025 Maximum Daily Flow	13,000 m^3/day

It was difficult to verify during the characterization program the current year flows, because the instrumentation for influent Parshall flume is not functioning.

The projected influent organic loading for the Mafraq Wastewater Treatment Facility is also based on the projections in the 2001 Conceptual Design Report. Table ES-1 provides a summary of the influent loading for the 20-year design period. These appear reasonable, and no changes are suggested at this point. Modularity and flexibility will be included in the conceptual design philosophy to accommodate some variations from these assumptions.

Table ES-1

Proposed Design Loads at Mafraq WWTP*

Year	Sewered Population	BOD_5		TSS		TN		TP	
		mg/L	kg/d	mg/L	kg/d	mg/L	kg/d	mg/L	kg/d
2005	25,786	925	1,985	925	1,985	156	335	54	116
2010	32,846	746	2,529	746	2,529	126	427	44	148
2015	41,135	708	3,167	708	3,167	119	535	41	185
2020	50,702	724	3,904	724	3,904	122	659	42	228
2025	61,557	724	4,740	724	4,740	122	800	42	277

*Source: 2001 Conceptual Design Report

Table ES-2 provides a summary of the effluent design criteria used for developing the recommended rehabilitation alternative. The effluent criteria are based on the 2002 Edition of the Jordanian Standard JS 893 for reclaimed wastewater discharge to wadis.

Table ES-2

Allowable Limits for Discharge of Treated Wastewater to Wadis or Water Bodies*

Parameter	Jordanian Standards JS 893 2002*	Units
Biological Oxygen Demand	60 **	mg/L
Total Suspended Solids	60 ***	mg/L
Chemical Oxygen Demand	150 ***	mg/L
Total Nitrogen	70	mg/L
Nitrate	45	mg/L
Dissolved Oxygen	≥ 1.0	mg/L
Turbidity	--	NTU
pH	6-9	-
E. Coli	1,000	colonies/100 mL
Intestinal Helminthes Eggs	≤ 1.0	eggs/L

**Source: 2002 Hashemite Kingdom of Jordan, Institution for Standards and Metrology, third edition, Jordanian Standards for Water & Reclaimed Domestic Wastewater, Section 5 (reference 23)
** Filtered BOD for wastewater systems with polishing reservoir
*** For biological treatment systems equipped with polishing reservoirs, the values are twice the given concentrations.

In summary, the rehabilitated wastewater treatment facility, serving the future (2025) sewered population of the Town of Mafraq will need to be designed based on the following design criteria:

- Average daily flow of 6,550 m^3/day
- Maximum daily flow of 13,000 m^3/day
- BOD_5 load of 4,740 kg/day
- TSS load of 4,740 kg/day
- TN load of 800 kg/day

The design will be based on plant effluent meeting the most recent Jordanian standards JS 893 2002 for discharge to wadis.

The nitrate concentrations in the effluent are limited to 45 mg/L (JS 893 2002), which is equivalent to 10 mg/L of nitrate-nitrogen. The impact of this standard on the design of the facility is reflected in the higher recirculation rates of the treated effluent to the denitrification units. For 45 mg/L of nitrates a recirculation rate of 115 percent will ensure full denitrification, while for 10mg/L, recirculation rate of 500 to 600 percent is required.

Recommended Rehabilitation Approach

The ideal wastewater management strategy for a community is one that, for its design period:

- Requires the lowest capital investment
- Minimizes operation and maintenance costs
- Can be operated by the available staff
- Reliably meets its discharge standards

For the upgrade to Mafraq WWTP, we considered only simple, low-tech natural processes requiring limited operator intervention. We examined also systems that will maximize the use of the existing ponds, either as they are, or with limited modifications.

One of the specific technologies we evaluated was the Advanced Integrated Pond System (AIPS) developed by Dr. Oswald of the University of California at Berkeley. After careful consideration we concluded that the AIPS is inappropriate for this upgrade, due to the following main reasons:

- Higher influent wastewater strength at Mafraq
- Limited number of AIPS installations around the world, and especially in developing countries
- The strict nitrate-nitrogen effluent standard of 10 mg/l

Waste Stabilization Ponds (WSP), Aerated Lagoons (AL), Constructed Wetlands (CW), and Intermittent Sand Filters (ISF), as well as the Recirculating Sand Filters (RSF) are often referred to as low-rate systems, which require little or no mechanical equipment. Often a combination of several low-rate systems is required to meet stringent effluent standards. When properly designed and operated, low-rate systems can produce a final effluent comparable to high-rate systems, which use mechanical equipment.

We recommend a liquid process treatment train (two trains operated in parallel) consisting of influent screening, a wet weather storage lagoon, submersible pumping station to lift the incoming effluent, sedimentation/thickening tanks, predenitrification basins, aerated lagoons, recirculating sand filters, constructed wetlands and treated effluent storage. The sludge treatment train will include an aerated sludge stabilization/storage lagoon for sludge equalization,, sludge drying beds for dewatering, and windrow composting cells for stabilization. Dewatered sludge from the sludge drying beds will need to be trucked to a landfill for final disposal. Composted sludge can be reused beneficially as soil amendment by the nearby farmers.

The proposed flow sheet integrates the majority of the existing structures, most of which were found to be in sound condition, during our visit to the facility in May 2005. The volume of the facultative ponds can be increased substantially by eliminating the dividing wall between two parallel ponds. Other treatment units can be kept intact, or modified in similar fashion, according to decisions made during the design development phase. Some berms may also be raised to augment the process capacity and add storage for periods of high flow.

The treatment units for the upgraded plant occupy more land than the existing plant. However, the expanded facilities do not encroach on the sites reserved for water reuse. The final size and configuration of process units will be developed during subsequent design phases of the project.

These recommendations should provide a cost-effective solution for upgrading the existing wastewater treatment facility to comply with Jordanian Standard JS 893 2002. The technologies selected are proven, reliable wastewater treatment processes that require minimal operator intervention. In situations with strict effluent standards a combination of proven technologies is the best choice, providing multiple treatment steps.

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List of Acronyms

The following are definitions of acronyms commonly used in this document.

AADF	Annual Average Daily Flow
AIPS	Advanced Integrated Pond System
AL	Aerated Lagoons
ASB	Aerated Stabilization Basins
BOD	Biochemical Oxygen Demand
CBOD	Carbonaceous Biochemical Oxygen Demand
CCP	Composite Correction Program
COD	Chemical Oxygen Demand
CPE	Comprehensive Performance Evaluation
CW	Constructed Wetlands
DB	Dosing Basin
EPA	Environmental Protection Agency
EB	Existing Disinfection

Ha	Hectare (10,000 square meters)
I/I	Infiltration/Inflow
ISF	Intermittent Sand Filters
mg/L	Milligram per Liter
MMADF	Maximum Month Average Daily Flow
MWI	Ministry of Water and Irrigation
NBOD	Nitrogenous Biochemical Oxygen Demand
O&M	Operational and Maintenance
RSF	Recirculating Sand Filters
RSS	Royal Scientific Society
SBOD	Soluble Biological Oxygen Demand
SCBOD	Soluble Carbonaceous Biochemical Oxygen Demand
SDB	Sludge Drying Bed
SL	Sludge Lagoon
SRS	Septage Receiving Station
TKN	Total Kjeldahl Nitrogen
TN	Total Nitrogen
TSS	Total Suspended Salts
UV	Ultra Violet
VSS	Volatile Suspended Solids
WAJ	Water Authority of Jordan
WSP	Waste Stabilization Pond
WWTP	Wastewater Treatment Plant

Section 1 Introduction

1.1 Background

The City of Mafraq, located in northern Jordan near the border with Syria, is served by the Mafraq Wastewater Treatment Plant (WWTP), which is situated approximately 6 km north of the city on a 37 hectare (ha) site. Wastewater from the city is conveyed to the plant through a 6 km 900 mm diameter gravity sewer. The plant, designed for an average daily flow of 1,850 cubic meters uses a series of 12 waste stabilization ponds to treat the wastewater generated by the town's current population of about 55,000 people. Approximately 24 ha of the site are leased to a local farmer who grows olives, wheat, sorghum and alfalfa. The treated effluent from the plant is pumped to the fields used by the farmer and is distributed in ditches for flooded irrigation using a border check system.

Since the facility was commissioned in 1988, the plant has experienced problems complying with the Jordanian Technical Regulations for reclaimed domestic wastewater. In an effort to resolve the treatment plant deficiencies, the Ministry of Water and Irrigation of Jordan, in cooperation with USAID, retained the services of a U.S. based engineering firm with the assistance from a Jordanian consulting engineering firm. The assignment involved preparing a feasibility study, conceptual design report, an environmental assessment and contract documents for a solution to bring the plant into compliance and meet future disposal requirements.

As part of their overall water management policy implementation program, the Ministry of Water and Irrigation (MWI) and the Water Authority of Jordan (WAJ) requested USAID's assistance in the assessment of the rehabilitation of the Waste Stabilization Pond (WSP) and possible financing arrangements for the upgrade or replacement of some components of the plant, Jordan, using low cost/low tech type technologies.

In an effort to correct the deficiencies at the Mafraq WWTP as soon as possible, USAID has added the assessment for upgrading this plant to an existing contract (Task Order No. EPP-I-800-03-00013-00) with IRG as prime contractor, ECODIT as lead implementing contractor, and Stearns & Wheler as engineering design subcontractor. The objective of this assessment is to assist the Water Authority of Jordan and USAID in identifying a lower technology, low maintenance solution that can be implemented to rehabilitate and expand the Mafraq WWTP.

1.2 Project Scope

The project scope is to determine the feasibility and technical viability of rehabilitating the existing Waste Stabilization Ponds and/or expanding (using low cost/low tech) to accommodate current and future loads and flows and to meet Jordanian Standards JS 893 2002 for discharge to wadis (reference 25).

1.3 Wastewater Treatment Plant Assessment Methodology

Prior to upgrading a WSP system, its performance should be evaluated, and its performance-limiting factors determined. Only then can appropriate decisions regarding corrective actions be made.

A number of approaches can be used to "troubleshoot" an existing WSP system, and identify methods to upgrade deficient systems. Our experience suggests that a sound approach is the Comprehensive Performance Evaluation/Composite Correction Program (CPE/CCP), developed by the U.S. Environmental Protection Agency (EPA) in the late-eighties (reference 15). We chose to employ this approach for this assignment.

The CPE/CCP approach involves an evaluation conducted in two phases:

Phase 1:	***Comprehensive Performance Evaluation (CPE)***
Phase 2:	***Composite Correction Program (CCP)***

The comprehensive performance evaluation phase is a thorough review and analysis of Mafraq's WWTP design capabilities and associated administration, operation, and maintenance practices. It is conducted to provide information for USAID/WAJ administrators to make decisions regarding efforts necessary to improve performance. The primary objective is to determine if significant improvements in treatment can be achieved without making major capital expenditures. This is accomplished by identifying and prioritizing those factors that limit performance and can be corrected to improve performance.

The composite correction plan phase involves a systematic approach prioritizing corrective actions, needed to eliminate those factors that limit performance. Its major benefit is that it optimizes the capability of existing facilities to perform better and/or treat more wastewater.

The CPE/CCP Methodology included the following steps.

Step 1:	Gather data about the plant from existing reports and studies.
Step 2:	Visit the WWTP facilities.
Step 3:	Meet with USAID and WAJ and validate initial impressions.
Step 4:	Develop and execute a wastewater characterization program.
Step 5:	Evaluate major unit processes.
Step 6:	Evaluate the characterization program results.
Step 7:	Identify performance-limiting factors.
Step 8:	Screen the rehabilitation alternatives.
Step 9	Recommend the most appropriate rehabilitation/upgrading alternative.

Step 10: Propose an action plan to design, build and implement the identified improvements to the plant as soon as possible.

The results from each major evaluation step are presented in subsequent sections of this report.

Section 2 Description of Mafraq Wastewater Treatment Plant

2.1 Facility Description

The Mafraq WWTP is a WSP system commissioned in 1988. The treatment facility, located approximately 6 km north of the City of Mafraq on a 37-ha site was designed for a flow rate of 1,850 m^3/day , organic load of 1,560 kg BOD_5 per day and solids loading of 1,700 kg TSS per day. Wastewater from the city is conveyed to the WWTP through a 6 km (900 mm in diameter) gravity sewer.

The existing wastewater treatment plant includes a mechanical screen, bypass manual bar rack, and a Parshall flume for influent flow metering followed by two identical biological treatment trains operated in parallel. Each treatment train includes one anaerobic lagoon, three facultative lagoons, and two maturation ponds, configured to be operated in series. Chlorine disinfection of the treated effluent was implemented as part of the initial design but is not currently operational. Sludge accumulation in the racetrack-shaped-chlorine-contact chamber is the main reason for the current non-operational status of the disinfection unit. The final effluent is conveyed by and land-applied to a series of agricultural fields that comprise 24 ha of the 37-ha site. These fields are leased to an independent farmer from the area.

2.2 Performance Problems

Since the facility was commissioned in 1988, the plant has not been able to comply with the Jordanian Standards for reclaimed domestic wastewater reuse. It is worth noting that discharge to the nearby wadis is not allowed currently by the Jordanian Water Authorities. Operating as a "Zero discharge type system" creates significant stress for the personnel and the facility.

The poor historical performance is primarily attributable to a severe organic overloading of the first stage facultative ponds. Currently, the majority of the ponds act as a series of anaerobic reactors, providing only partial treatment of the influent waste. Solids carryover, as a result of the poor treatment and the resulting organic loading on the next stages of the WSP system, has resulted in solids accumulating in all subsequent treatment units, including the chlorine contact basin.

In addition, USAID indicated that the operation and maintenance of the WWTP faces several operational and institutional challenges, specifically:

- Limited financial resources.
- Lack of skilled human resources.
- Lack of basic facilities at the plant, such as communications, a laboratory and suitable accommodations for the workers.
- The need to match the plant output with the irrigation needs of the farmers.
- Lack of reliable flow measurement facilities at the inlet and outlet of the plant.
- Lack of maintenance for most of the treatment plant units such as gates between the ponds, the chlorination unit and others.

Other operational difficulties, reported by the plant staff, include the head works. The existing mechanical screen was inoperable in May 2005, thereby requiring manual removal of the screenings (the screen was put back in operation recently). The instrumentation for the influent flow meter does not work well. Therefore, the influent flow rates and subsequent organic loading rates applied to the facility, based on historical data, may not be accurate. The operational manual does not take into account either the physical characteristics of the facility, or the influent organic and solids characteristics, leaving limited operational choices to the operators of the plant.

2.3 Historical Flows and Loads

The historical flows and loads for the facility were obtained from the 2001 Conceptual Design Report. The data presented in the Design Report indicates an estimated water use per capita of 97 liters per day. Table 2.3-1 provides a summary of the averaged historical plant data from 1988 to 1999. The data presented in the table also shows that the Mafraq WSPs failed to provide adequate treatment from the time the plant was put into operation in 1988. The effluent concentrations of BOD, TKN, Fecal Coliforms and COD were below the earlier editions of the Jordanian standards JS 893. For example Table 2.3-1 shows the historical values of the effluent BOD since 1988. The effluent concentration was never in compliance with the standards (allowable BOD is 60 mg/L, while the reported values are between 72 and 284 mg/L). The nitrogen concentration also exceeded consistently the allowable limits mainly because the system was never able to maintain healthy nitrifying population in the ponds. On the other hand the pathogen destruction was also inefficient and influent and effluent fecal coliforms levels exceeded consistently the effluent standards.

Several of the consulted documents, including the 2001 Conceptual Design Report, explain the poor performance by citing increasingly high flows and organic concentrations. In our opinion, these are only partial explanations. The reported concentrations, when correlated with higher-than-design flow values, still result in organic loadings within the initial design values (see Table 2.3-1), which were:

- Average flow of 1,850 m^3/day.
- Average BOD_5 of 1,563 kg/day (design concentration of 845 mg/L).
- Average TSS of 1,702 kg/day (design concentration of 920 mg/L).

Table 2.3-1

Historical Performance Data from 1988 to 1999

Paralleled with the Calculated Average BOD Loading

Year	Influent Plant Flow (m^3/day)	Influent BOD_5 (mg/L)	Effluent BOD_5 (mg/L)	Allowable BOD_5 (mg/L)	Effluent TSS (mg/L)	Allowable TSS (mg/L)	Effluent NH_4-N (mg/L)	Allowable TN (mg/L)
1988	782	634	114	60	293	60	-	70
1989	950	745	270	60	217	60	-	70
1990	1,091	702	72	60	184	60	-	70
1991	1,395	960	225	60	251	60	-	70
1992	1,340	694	257	60	171	60	-	70
1993	1,377	970	249	60	199	60	-	70
1994	1,317	610	246	60	199	60	-	70
1995	1,290	868	270	60	181	60	144	70
1996	2,379	641	284	60	202	60	184	70
1997	2,638	564	200	60	175	60	155	70
1998	2,297	714	250	60	211	60	140	70
1999	1,933	566	198	60	249	60	142	70

Table 2.3-2 presents the range of wastewater characteristics from 1995 to 1999. The data reported in Table 2.3-2 is based on weekly grab samples and as such, may only represent an approximation of the influent characteristics.

It is obvious that each pollutant, recorded in Table 2.3-2 varied substantially throughout the reported period (1995 to 1999), indicating either a strong industrial contribution, or conditions leading to high variability (long sewer line, changing population, inadequate sampling, etc). Such influent variability prompts the use of robust processes, capable of supporting such variable loads.

Table 2.3-2

Influent Wastewater Characterization from 1995 to 1999*

Parameter		Year				
		1995	**1996**	**1997**	**1998**	**1999**
BOD_5 mg/L	Minimum	459	480	347	230	232
	Average	868	641	564	714	566
	Maximum	1,600	1,877	644	2,814	1,734
COD mg/L	Minimum	851	812	807	720	436
	Average	1,759	1,427	N/A	1,110	1,358
	Maximum	3,453	5,218	1,853	1,660	2,971
TSS mg/L	Minimum	206	288	273	245	164
	Average	967	837	577	452	424
	Maximum	2,947	1,044	1,317	905	1,014

*Source: 2001 Conceptual Design Report

The data indicates that influent BOD_5 concentration varied between 230 mg/L and 2,814 mg/L with an average concentration of 671 mg/L. The influent COD variation was between 720 and 5,218 mg/L, with a COD: BOD ratio of about 2:1, which is typical of domestic wastewater. The TSS concentrations varied between 164 mg/L and 2,947 mg/L, a sign of rather strong wastewater, and an initial indication of the possibility of increased volatile suspended solids (bacterial biomass), created in the long influent piping between the City and the WWTP.

Table 2.3-3 provides the average nutrients concentrations from 1995 to 1999. The validity of the values reported in Table 2.3-3 could be questioned, due to the grab type samples used to characterize the raw wastewater.

Table 2.3-3

Mafraq WWTP Influent

Ammonia-Nitrogen and Ortho-Phosphorus Data from 1995 to 1999*

Parameter	Year				
	1995	**1996**	**1997**	**1998**	**1999**
NH_4-N, mg/L	144	184	138	141	111
PO_4-P, mg/L	73.8	84.3	43.7	79	28.5

*Source: 2001 Conceptual Design Report

2.4 Future Flows and Loads

The discussion on flows and loads presented in this section is based on the information provided in the 2001 Conceptual Design Report. As will be shown later in this report, some of the design values for TSS may be higher than TSS values calculated based on population alone. They could be adjusted for the final design and after a consultation with the WAJ.

According to the Design Report, the flow projections were developed using World Bank population growth estimates, per capita water estimates, and assumptions of wastewater collection rates based on sewerage of new housing areas. Table 2.4-1 provides the 2001 summary of the projected wastewater flow rates.

Table 2.4-1

Projected Wastewater Flows*

Year	Population Estimate	Percent Sewered	Sewered Population	Water Consumption (m^3/cap-day)	% Captured Flow	Flow Rate (m^3/day)
2005	46,883	55	25,786	0.104	80	2,145
2010	54,743	60	32,846	0.129	80	3,390
2015	63,285	65	41,135	0.136	80	4,476
2020	72,432	70	50,702	0.133	80	5,395
2025	82,076	75	61,557	0.133	80	6,550

* Source: 2001 Conceptual Design Report

The annual average design flow rate projected for the design year (2025) is 6,550 m^3/day. However, as the authors of the previous studies indicate, treatment facilities are typically designed for the maximum month average daily flow (MMADF) to ensure proper performance and compliance with effluent standards. According to the 2001 Conceptual Design Report, the 1995 plant data is the most reliable influent flow data available for quantifying the ratio between maximum month average daily flow (MMADF) and the annual average daily flow (AADF). In 1995, the AADF and MMADF were 1,297

m^3/day and 1,431 m^3/day, respectively. This results in a ratio for the MMADF to AADF of approximately 1.10.

Historical flow data was reviewed for other plants in Jordan to determine the MMADF: AADF ratio and compared to the ratio for Mafraq. The ratios ranged from 1.06 to 1.67 with an average value of 1.17. A MMADF:AADF of 1.40 was recommended in the 2001 Conceptual Design Report (retained for this evaluation), based on the reliability and condition of the existing instrumentation for the influent Parshall flume and review of historical flow data for other treatment facilities in the region. We believe that the previous studies provided sufficient information to the regulatory authorities, who examined and approved those design criteria. We have thus retained these design values for our study.

Peak hourly hydraulic flows were also evaluated in the 2001 Conceptual Design Report. The peak flows were considered to ensure that sufficient freeboard is provided for treatment units, and that the conveyance piping and pumping systems are sized properly. According to the 2001 Conceptual Design Report, it is not uncommon for peak wet weather flows in Jordanian towns to be approximately three to five times the AADF. Since limited influent flow rate data is available, a mid-range value of four times the AADF was selected for the year 2025 peak hourly flow. Therefore, the plant hydraulics will be based on a peak hourly flow of:

- 26,200 m^3/day for the influent screening facilities
- 13,000 m^3/day for the remaining components of the plant

Screened influent, exceeding a maximum daily flow of 13,000 m^3day would be diverted and stored in a wet weather storage basin and processed during low flow periods. This would eliminate the need to oversize the subsequent treatment units.

The projected influent organic loading for the Mafraq Wastewater Treatment Facility will be based also on the proposed design values found in the 2001 Conceptual Design

Report. Table 2.4-2 provides a summary of the pollutant loading for a 20-year design period.

Table 2.4-2

Design Loads at Mafraq WWTP*

Year	Sewered Population	BOD_5		TSS		TN		TP	
		mg/L	kg/d	mg/L	kg/d	mg/L	kg/d	mg/L	kg/d
2005	25,786	925	1,985	925	1,985	156	335	54	116
2010	32,846	746	2,529	746	2,529	126	427	44	148
2015	41,135	708	3,167	708	3,167	119	535	41	185
2020	50,702	724	3,904	724	3,904	122	659	42	228
2025	61,557	724	4,740	724	4,740	122	800	42	277

*Source: 2001 Conceptual Design Report
Notes: Calculated based on 77 g BOD/capita/day; 77 g TSS/capita/day; 13 g TN/capita/day; 4.5 g TP/capita/day. Does not take into account the higher TSS values recorded during the 2005 characterization program in May, which can be modified during the detailed design.

Table 2.4-3 provides a summary of the effluent design criteria used for developing the recommended rehabilitation alternative. The effluent criteria are based on the 2002 Edition of the Jordanian Standard JS 893 for reclaimed wastewater discharge to wadis.

Table 2.4-3

Allowable Limits for Discharge of Treated Wastewater to Wadis or Water Bodies*

Parameter	Jordan Standards JS 893 2002*	Units
Biological Oxygen Demand	60 **	mg/L
Total Suspended Solids	60 ***	mg/L
Chemical Oxygen Demand	150 ***	mg/L
Total Nitrogen	70	mg/L
Nitrate	45	mg/L
Dissolved Oxygen	≥ 1.0	mg/L
Turbidity	--	NTU
pH	6-9	-
E. Coli	1,000	colonies/100 mL
Intestinal Helminthes Eggs	≤ 1.0	eggs/L

*Source: 2002 Hashemite Kingdom of Jordan, Institution for Standards and Metrology, third edition, Jordanian Standards for Water & Reclaimed Domestic Wastewater, Section 5 (reference 23)
**For biological treatment plants with polishing ponds, BOD is considered as the filtered BOD
***For biological treatment plants with polishing ponds, the allowable limits are twice this number

Section 3 Comprehensive Performance Evaluation

Much of lagoon (pond) troubleshooting depends on correct diagnosis of the problem and an understanding of wastewater pond ecology. Optimization of a Waste Stabilization Pond System depends upon a thorough understanding of what is happening physically, chemically, and biologically in the ponds.

When there are changes in lagoon water chemistry due to loading, dissolved oxygen, temperature, sunlight or other influences, there are corresponding changes in the pond's microbial ecosystems. These changes alter the water quality and, in turn, affect significantly the capability of the system to reduce the pollutants efficiently.

The diagnosis of Mafraq Waste Stabilization Pond System was conducted in five steps, described in details below. To facilitate the understanding by the reader, we have included also a general description of the main processes taking place in a WSP system.

The five "diagnostic evaluation" steps were:

1. Physical inspection and measurement of the pond's dimensions on the site (May 2005), validation of the measurement with WAJ's "as built drawings" for the facility, and comparison of the results with plant's references (Mafraq monthly reports to WAJ, 2003 Draft Design Study Reports).
2. Comparison of historical loadings with the initial design criteria and WSP design practices
3. Treatment capacity evaluation of each unit process.
4. Evaluation of the wastewater characterization program results
5. Identification of the major performance limiting factors.

3.1 Waste Stabilization Ponds Process Functions

A Waste Stabilization Pond system is usually comprised of three unit processes:

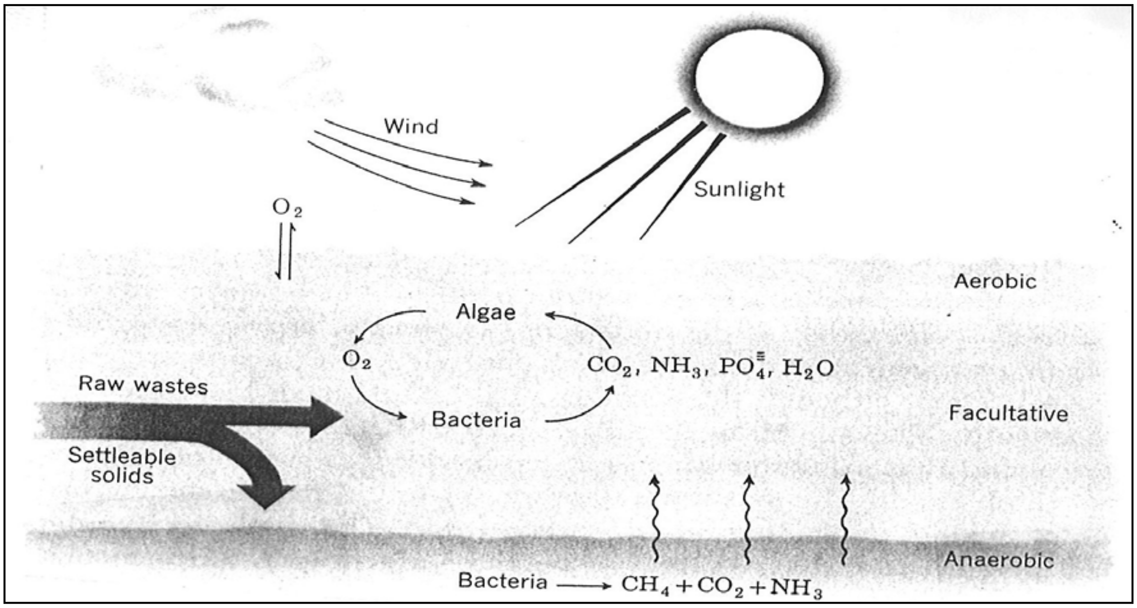
- Anaerobic pond(s)
- Facultative pond(s)
- Maturation pond(s)

The anaerobic ponds are usually included as a pretreatment step in any system receiving wastewater with high organic concentrations. Because of low average water use, the wastewater in Jordan is stronger than the typical values for average strength domestic wastewater of about 250 to 300 mg of BOD_5 per liter. The anaerobic ponds are usually designed based on volumetric organic loading, and their proper operation will continue as long as their treatment volume is not reduced by solids accumulation. Sometimes additional treatment volume is added to take into account less frequent sludge removal. In Mafraq, the sludge removal has occurred every 9 years (1996, 2005), and the anaerobic ponds volume occupied by the solids entering the plant has reached 50 percent (May 2005). When significant volume reduction occurs, the anaerobic degradation efficiency is reduced, and more and more of the incoming organic load overflows to the next treatment step: the facultative ponds.

The Mafraq anaerobic ponds also served as solids stabilization reactors, as well as hydraulic equalization tank in periods of high flow.

The ***facultative pond***, or ponds in series, is the next component of the WSP process.

The following diagram illustrates the facultative pond processes. Three regimes may be found in a healthy facultative pond: the pond surface (the aerobic zone), the middle of a pond (the facultative zone), and the bottom or anaerobic portion of a pond. Each section of the pond has an important function to perform.



THE POND SURFACE – THE AEROBIC ZONE

The upper surface of a facultative pond is responsible for:

- Pathogen destruction by UV
- Odor control
- Nutrient removal
- Metals removal
- Reaeration
- BOD_5 removal

Algae thrive in the upper 50 centimeters of the surface of a wastewater pond. They cause the pH to rise by consuming CO_2 and then bicarbonate ion, the buffering capacity of the pond. Elevated pH is important for several reasons. A high pH;

- Causes the dissociation of the hydrogen sulfide molecule. This means that if the pH rises above 8.2, the H_2S molecule will dissociate into its components parts. At pH above 8.2, hydrogen sulfide cannot exist as an odor-producing molecule.
- High pH encourages pathogen inactivation and dye-off.
- Elevated pH is important for the volatilization and removal of ammonia from the lagoon system.

Algae also add oxygen to the system. As long as about 2 mg/L of oxygen covers the surface of the pond, there will be enough oxygen to oxidize any emerging odor. Effluent recirculation is also usually part of the design, adding highly oxygenated effluent from the maturation ponds to the front part of the facultative ponds. During the inspection of Mafraq WSP in May 2005 it was noted that water reuse was so high that effluent recirculation could not be practiced because water levels were very low (the plant is "Zero discharge").

THE MIDDLE OF A POND – THE FACULTATIVE ZONE

The middle portion of a facultative pond is responsible for:

- Odor control
- Denitrification
- BOD_5 removal
- Phosphorus removal

THE BOTTOM OR ANAEROBIC PORTION OF A POND

The bottom portion of a facultative pond is responsible for:

- BOD_5 removal
- Sludge storage
- Sludge digestion
- Denitrification
- Pathogen destruction
- Metals and nutrient storage

Retention time is critical to pond performance because many of the pond's stabilization processes require contact time between the biomass and the many biological and chemical stabilizing influences that exist in a pond.

Factors such as predatory forces, ultraviolet radiation, competitive exclusion, starvation, sedimentation and natural die-off all require time. Because of this, BOD reduction, pathogen control, TSS problems, and inefficient nutrient removal are closely linked to a pond retention time. BOD removal also cannot take place under very high surface organic loadings (in Mafraq the applied BOD loadings exceeded the recommended by a factor varying from 4 to 10 times).

In addition, many WSP in use today were designed at a time when the principles of lagoon hydraulics were poorly understood. Today such design is part of the state-of-the art practices available to wastewater treatment professionals. This is important because so much of a facultative pond's ability to stabilize wastewater is dependent upon that wastewater's exposure to the stabilizing influences of a pond. Experience has shown (EPA manual on POTW performance limiting factors) that "far too many lagoons in use today have low retention times because of short-circuiting".

3.2 Physical Inspection of the WWTP

The wastewater treatment plant was first visited by Stearns & Wheler staff on May 21, 2005. Figure 3.2-1 presents the overall view of the treatment units together with the measured dimensions of the ponds. The measured dimensions were subsequently compared with the dimensions given in the "as built drawings" provided by the Water Authority of Jordan. The measured dimensions are consistent with the dimensions of the ponds found on the "as built drawings". Those dimensions were then compared with values found in three additional sets of documents:

- Mafraq WWTP monthly reports to WAJ.
- 2001 Conceptual Design Report.
- The RFP issued for the assessment of the rehabilitation potential of the plant.

All three sources cite that "Mafraq WWTP is a Waste Stabilization Pond System with twelve (12) one-half hectare (0.5 hectares) surface area ponds each". This is not the case; they are considerably smaller than this, as noted in Figure 3.2-1.

The smaller surface areas also result in smaller volumes of each treatment unit, providing shorter hydraulic retention times and resulting in less efficient treatment.

Table 3.2-1 summarizes the total measured surface areas and volumes and compares them with the calculated surfaces and volumes based on documented pond size of 0.5 hectares (each pond). The anaerobic ponds are approximately half of the 0.5 ha area and volumes; the facultative ponds approximately 30 percent smaller, as well as the maturation ponds. The current hydraulic retention time in the system is about 44 percent of what would have been with pond surface area of 0.5 ha.

Table 3.2-1

Mafraq WSP Pond Dimensions

Description		Surface Area (m^2)		Treatment Volume (m^3)	
		Measured on the Site	Documented as Existing	Calculated with Measured Surface area	Calculated with 0.5 ha of surface area/pond
Total Area of	Anaerobic	5,700	10,000	13,140	26,980
	Facultative	21,300	30,000	27,348	42,162
	Maturation	~11,000	20,000	9,880	20,240
HRT at flows of	2,000m^3	-	-	~25 days	~45 days
	6,550m^3	-	-	~7.7 days	~14 days
	10,000 m^3/d	-	-	~5 days	~9 days
	26,000 m^3/d	-	-	~2 days	~3.5 days

As discussed in the previous section, reduced surface areas and volumes of each treatment unit have significantly affected the treatment system since the beginning of operations in 1988. Accumulated solids and non biodegraded BOD in each treatment stage, has prevented the treatment units from functioning properly ever since the initial operation of the plant. The problem has been exacerbated over the years as solids have continued to accumulate in the ponds, reducing the effective treatment volumes.

Insert Figure 3.2.1

As it will be seen later in this section, the reduced size of some treatment units will have greater impact on the performance of the plant than other units (facultative and the maturation ponds, for example, are designed based on surface area loadings, while the anaerobic ponds design is based on volumetric organic loading).

Table 3.2-2 summarizes the calculated current organic loadings and compares them with the suggested maximum organic loadings found in the Design Manual for WSP, written by one of the leading lagoon technology experts, Dr. Duncan Mara from Leeds University in the United Kingdom.

Table 3.2-2

Historical Organic Loadings

Year	Influent Plant Flow (m^3/day)	Influent BOD_5 (kg/day)	Design BOD_5 (kg/day)	Recommended* Organic loading (kg BOD/ha/day) Winter/Summer	Current Organic Loading to F 11 & F 21* Kg BOD/ha/d
1988	782	496	1,560	110/180	870
1989	950	798	1,560	110/180	1,400
1990	1,091	765	1,560	110/180	1,342
1991	1,395	1,340	1,560	110/180	2,351
1992	1,340	930	1,560	110/180	1,632
1993	1,377	1,336	1,560	110/180	2,344
1994	1,317	803	1,560	110/180	1,410
1995	1,290	1,120	1,560	110/180	1,965
1996	2,379	1,524	1,560	110/180	2,674
1997	2,638	1,488	1,560	110/180	2,610
1998	2,297	1,640	1,560	110/180	2,877
1999	1,933	1,094	1,560	110/180	1,920

* D. Mara - WSP Design Manual

Comparison of the last two columns in Table 3.2-2 reveals that a primary reason for the poor performance is the excessively high organic loading to the first stage facultative ponds. The last column in Table 3.3-2 does not account for the benthic feedback of BOD that could have been occurring since the initial years of operation, due to the long gravity sewer line between the City and the plant. Usually a significant BOD reduction is observed in such lines, and this is seen in the May 2005 characterization program results.

Each train was designed to be operated in a series. Piping and valving that would have allowed the facultative ponds within a train to operate in parallel was not provided in the initial design. Such piping and valving could have helped minimize the overloading condition by spreading out the load. However, the facultative ponds of Mafraq, being undersized since the initial phases, have been overloaded since 1988, and have not provided adequate conditions for the establishment of the normal microbiological population required for efficient treatment of the wastewater.

During the plant visit in May 2005, the plant operating personnel indicated that flow distribution is not adequate, leading to uneven partition of the flow between the two parallel trains.

Organic overloading of the facultative ponds has resulted in a conversion of the various ponds into anaerobic reactors, suggested by the “blackish appearance” of the ponds observed during site visit.

When the normal microbiological processes typically observed in a healthy facultative pond cannot be established, the organic content of the waste trickles to the second stage facultative ponds. They, in turn, are overloaded, and lose their ability to treat the organic content of the waste, and the overloading cascades downstream.

The anaerobic ponds, being cleaned only every 9 to 10 years, also exacerbated the problems, by providing smaller volumes for treatment each year. In addition, as noted in the next section, benthic feedback of organics is very significant (almost double the filterable BOD during the characterization period), increasing further the overloading problems of the first stage facultative ponds. Total BOD_5, leaving the anaerobic ponds is increased by 30 percent compared to the influent values (probably due to high loss of organic solids), while filtered BOD is increased by 100 percent over the influent values, a clear indication of benthic feedback.

If benthic feedback is taken into account, then the overload to the first stage facultative ponds could be several times higher than suggested design values summarized in the last column of Table 3.2-2.

The maturation ponds were in better shape than the preceding treatment processes. However, they showed signs of sporadic organic overloading seen in the blackish traces of dried sludge on their banks.

Finally, current influent feed piping into each treatment cell, is located in the center of the short side of each pond and the discharge location is on the opposite side of the ponds. Such location of the influent and discharge piping is not recommended today. Current authoritative WSP design manuals recommend that inlet and discharge pipes be located in opposite corners of the ponds. The current arrangement may be promoting short circuiting, further decreasing the hydraulic retention time in each pond. A tracer study could be used to verify this hypothesis. "State-of-the art" design manuals for Waste Stabilization Ponds recommend "opposite corners" location of these inlet/outlet pairs.

3.3 Waste Stabilization Ponds (WSP) Design Criteria

According to D. Mara, Waste Stabilization Ponds must be sized based on the organic loadings summarized in Tables 3.3-1, 3.3-2 and 3.3-3. D. Mara suggests limiting BOD loadings to the anaerobic ponds to less than 300 g/m^3, for reduced odor generation potential, as well as for more consistent organic degradation.

Table 3.3-1

Design Loadings for BOD Removals in Anaerobic Ponds (D. Mara)

Design Temperature (T) (degrees C)	Volumetric BOD Loading	BOD removal (%)
< 10	100 grams BOD/m^3/day	40
10 - 20	20 T – 100 grams BOD/m^3/day	2T + 20
> 20	300 grams BOD/m^3/day	60

Facultative ponds need to be sized based on the surface organic loadings, summarized in Table 3.3-2.

Table 3.3-2

Suggested Design Loadings Facultative Ponds (D. Mara)

Design Temperature T (degrees C)	Surface BOD Loading
< 10	100 kg BOD/hectare/day
10 – 20	10 T kg BOD/hectare/day
> 20	$50 (1.072)^T$ kg BOD/hectare/day

Maturation ponds must be sized based on the surface organic loadings, summarized in Table 3.3-3

Table 3.3-3

Suggested Design Loadings Maturation Ponds (D. Mara)

Design Temperature T (degrees C)	Surface BOD Loading
< 10	50 to 75% of Values for Facultative Ponds
10 – 20	50 to 75% of Values for Facultative Ponds
> 20	50 to 75% of Values for Facultative Ponds

3.4 Current Sludge Management Practices

Sludge accumulates in the waste stabilization ponds, mainly in the anaerobic ponds, and is periodically removed by a hired contractor. The plant does not have any dedicated sludge removal equipment, as well as conditioning, thickening or dewatering facilities.

During the inspection of the plant in May 2005, the second treatment train was isolated, and the anaerobic pond A-2 had been drained and left to dry. The dried sludge volume was approximately 1,500 cubic meters, as measured by the Irbid wastewater management personnel in charge of evaluating the quantities of sludge in the ponds.

Photo of Anaerobic Pond A-2, Drained and Left to Dry

The anaerobic pond A-1 had a layer of scum on the surface. WAJ was in the planning process for the cleaning of the ponds. The sludge volume in A-1 was measured to be approximately 2,800 m^3, or 43 percent of the total anaerobic volume available for treatment. The characterization program results indicated that the solids content was about 9 percent (90,000 mg/L) in the operational pond, while the solids content in the pond A-2, which had been drained and was left to dry, was estimated at about 25 percent (250,000 mg/L).

Plant personnel communicated serious concerns about sludge management practices at the plant, and expressed the desire to have more flexible systems and easier sludge removal options. The previous anaerobic pond cleaning occurred 9 years ago. The accumulated volume of sludge (for nine years) in both ponds (before dredging/decanting) was calculated at about 6,000 cubic meters (approximate mass of 500,000 kg).

We attempted to compare the sludge that would have resulted from the reported TSS values with the volume of accumulated sludge in the basins. The mass of measured, accumulated solids is smaller than the actual TSS contribution. Using the average TSS content of the wastewater it was estimated that about 3,000,000 kilograms of solids have entered the WSP system in the last nine years. In comparison, the measured mass of solids was estimated at about 500,000 kilograms. We speculate that part of the difference may be solids leaving the plant in the effluent. In addition such difference suggest high levels of organic degradation in the ponds (about 85 percent for nine years), but also some possible inconsistencies with TSS measurements and reporting.

3.5 Mafraq Climate Data

Evaluation of the regional climatic conditions was done to document colder periods of the year, when decreased treatment efficiencies of the WSP are observed.

Table 3.5.1 summarizes meteorological data for the Mafraq region. The data is based on information provided in the 2000 Jordanian Climatological Handbook (reference 6).

Table 3.5-1

Mafraq Climatological Data

Month	Mean Air T°	Estimated Water Temp +3° winter -3°summer	# days with T° >30	# days with T° <0	# days with ground Frost	Total # Rainfall Days	Total Evap.
January	7.4	10.4	0	26.9	18.8	35.4	63.7
February	8.7	11.7	0	22.9	14.6	30.6	82
March	11.4	14.4	0.1	16.9	10.3	285	134
April	15.9	16	4	5.4	3.4	8.5	213.9
May	20.1	17.1	11.9	0.6	0.6	3.1	299.8
June	22.8	19.8	20.7	0	0	0	341.6
July	24.4	21.4	27.2	0	0	0	375.9
August	24.4	21.4	28	0	0	0	329.4
September	22.8	19.8	18.7	0	0	0.3	264.4
October	19.3	19.3	8.4	0	0.1	7.1	190.5
November	13.6	16.6	0	1	4.8	19.8	114.8
December	8.9	11.9	0	4	14.9	28.5	65.9

We analyzed the data provided in Table 3.5-1 to establish distinctions between summer and winter periods for design purposes. It appears that such differences do exist, and should be considered as an important design parameter.

This distinction is very important for wastewater treatment plants using the waste stabilization ponds technology, as well as other lower technology processes. Water and air temperatures will affect significantly the treatment performance of the plant. Different operational strategies are common in areas with significant temperature differences between seasons, and they will need to be a part of the design and future operations of a WSP process. If more compact process is selected by the designers, then the climatic effect is less significant. The following table summarizes the allowable organic loadings on the anaerobic, facultative and maturation ponds in the summer and winter seasons.

The values in the table have been calculated with the climatological data for the area, showing the difference between allowable loading in the summer and the winter months of the year. Operating WSP in such climate is usually done by using parallel trains in the winter, and converting the ponds to operation in series in the summer months.

Table 3.5-2

Recommended Seasonal Organic Loadings for WSP Located in Mafraq

Organic Loading	Units	Summer			Winter		
		A*	F	M	A	F	M
BOD_5	kg/m3/d	270	---	---	126	---	---
	kg/ha/d	---	180	135	---	110	80
	Estimated removal efficiency	~56%	~ 85%	~ 90%	~42%	~75%	~80%

*A = Anaerobic ponds; F = Facultative ponds; M = Maturation ponds

In other words, if the expanded facility relies on lagoon type process, using the winter conditions as the governing design criteria, the main components of a WSP plant must have the following dimensions:

- Anaerobic ponds: volume of about 15,000 m^3
- Facultative ponds: Surface area of about 350,000 m^2
- Maturation ponds: Surface area of about 140,000 m^2

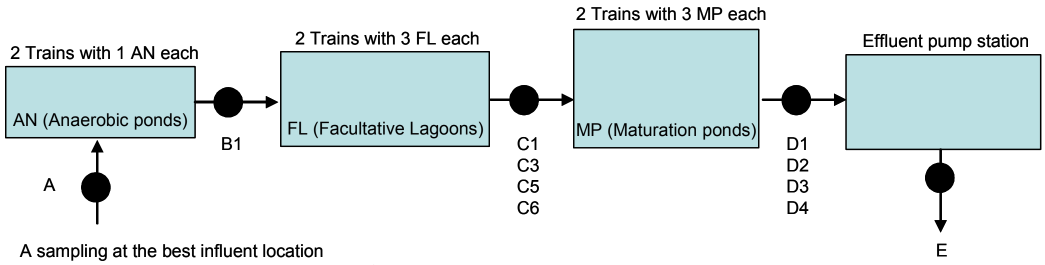
3.6 Wastewater Characterization

A comprehensive wastewater characterization program was planned and executed in May 2005. Sampling (grab and composite) of five wastewater streams and in the ponds was undertaken out by the Royal Scientific Society. The samples were collected from the influent wastewater, as well as the effluent side of the anaerobic ponds, the facultative ponds, the maturation ponds, the final effluent and the ponds themselves.

Due to physical constraints, some of the samples were grab rather than composite. Sampling of the accumulated solids in the ponds was carried out from the berms of each lagoon. The sampling locations are illustrated in Table 3.6-1 together with the list of parameters analyzed during the sampling program in May 2005.

Table 3.6-1

Analytical Testing for Mafraq WWTP Assessment



A sampling at the best influent location
B sampling in the pipe/channel between AN & FL
C sampling in the pipe/channel between FL & MP
D sampling in the effluent pipe/channel
AN,FL, MP – Sludge Samples from the center (TBD) of the lagoons (measured in mg/kg dry)
Sludge depth measured from a boat and taken in ~9 locations in each AN, FL and MP – TBD on site

Parameter	A	B	C	D	E	AN	FL	MP
BOD_{5t}*	●				●			
BOD_{5f}*	●				●			
COD	●				●			
TSS	●				●	●	●	
TVSS						●	●	●
TDS						●	●	●
TS						●	●	●
TVS						●	●	●
TKN	●				●			●
Ammonia	●				●			
H_2S	●	●	●	●	●			
Alkalinity	●				●			
DO+Temp							●	●
Metals in sludge						●	●	●

** See figure 3.6-1 for full list of analyzed parameters*

In order to identify other potential sources of treatability problems throughout the plant, BOD analyses covered the following species:

- **Regular Five Day Biochemical Oxygen Demand in 5 days (BOD_{5t}).** A standard used to measure the strength of wastewater. BOD_5=$CBOD_5$+$NBOD_5$. Used as a standard, also used as a testing starting point to understand more about what is going on in a lagoon system. The BOD_5 test is needed to calculate $NBOD_5$, an indication of a lagoon's ability to nitrify.
- **Filtered Biochemical Oxygen Demand (BOD_{5f})** Also called a Soluble BOD_5. In this test the BOD_5 test sample is first run through a TSS filter. The

$SBOD_5$ is the most readily oxidizable portion of the wastewater sample. $SBOD_5=SCBOD_5+SNBOD_5$. $SBOD_5$ is used to calculate $SCBOD_5$**.**

- **Carbonaceous Biochemical Oxygen Demand ($CBOD_5$).** $CBOD_5$ is the BOD_5 test run with a nitrification suppressant added to inhibit nitrification's effect on dissolved oxygen in the BOD_5 test bottle. $CBOD_5=BOD_5-NBOD_5$. $CBOD_5$ is a better measure of a lagoon's ability to stabilize waste.
- **Nitrogenous Biochemical Oxygen Demand ($NBOD_5=BOD_5-CBOD_5$)**. $NBOD_5$ represents the relative number of nitrifying bacteria in the BOD test bottle.
- **Soluble Carbonaceous BOD_5 ($SCBOD_5$).** The BOD_5 test is run after it is filtered and the nitrification suppressant is added. $SCBOD_5$ reveals the influence of a lagoon's sludge blanket in feeding BOD back to the liquid interface (benthic feedback). This measure gave important information about benthic release of BOD_5 back into solution.

The following figures summarize the results of May 2005 Wastewater Characterization Program:

Figure 3.6-1 Sampling and analyses of influent and effluent of each treatment process.
Figure 3.6-2 Algae count and type of algae detected.
Figure 3.6-3 Composite influent/effluent sampling and sludge characterization.

Insert Figure 3.6-1

Insert Figure 3.6-2

Insert Figure 3.6-3

As it can be seen from Figure 3.6-1, the influent soluble organic content (BOD Filtered) is significantly lower than the average Jordanian wastewater strength of about 650 mg/L. The 2005 characterization program shows that close to 80 percent of the influent wastewater BOD is in particulate form and only 20 percent in the readily biodegradable soluble form. This is the most likely due to organic degradation occurring in the gravity sewer line between the City and the plant. Such condition simply eliminates the main reason for the anaerobic pond in a WSP system: minimize the BOD load to the subsequent sections. In this project the anaerobic ponds have likely operated as sludge digesters.

The high volatile suspended solids concentrations observed in the influent wastewater may be explained by the high retention time in the gravity sewer between Mafraq and the plant. A 6 km sewer line of 900 mm, flowing half full, provides a volume equivalent to about 23 hours retention time at the current average flow of 2,000 m^3/day. The sewer line acts as a hybrid anaerobic reactor, degrading a significant portion of the influent BOD_5. The high concentrations of volatile solids in the influent are in fact probably biological matter (bacterial cells), produced during the degradation of the BOD_5.

The high TSS content in the influent wastewater would significantly affect the size of all subsequent unit processes, if not removed promptly after reaching the facility. Two main options were examined as possible TSS handling alternatives: one accumulating the solids in anaerobic ponds, and then periodically removing the sludge. The second option is to remove the solids as soon as they reach the plant with non-mechanized settling process such as sedimentation/thickening tanks. This second option was retained for the proposed rehabilitation alternative.

Even if larger anaerobic ponds might seem to provide higher volume for sludge storage, and perhaps higher degree of sludge stabilization, we felt that all subsequent treatment units would have to be significantly larger due to a phenomenon called benthic release of BOD_5. Benthic release occurs when suspended solids settle and dead microorganisms accumulate and a solids layer builds up on the bottom of the lagoons. This layer is decomposed by anaerobic and facultative organisms over time. This process releases organic acids and increases the wastewater BOD. Operating experience has shown that this release is often highest in the early spring after the anaerobic bacteria become active. After the colder periods of the year some “turn over” of the bottom of the ponds can occur, increasing even further the load into the overlaying water of the ponds. This release can be a significant load on the treatment system at a time when biological activity is low and other factors are causing stress on the system (e.g., high infiltration slowly increasing temperatures, etc.)

We recommend installing a static settling system, such as sedimentation/thickening tanks, as the first treatment step after the screening and influent pumping. Sludge would be removed hydrostatically and directed either to a sludge storage lagoon (can be either an earthen basin or concrete tank system) or directly to the sludge drying beds. Sludge drying beds produce stabilized sludge, in compliance with the existing standards for beneficial reuse of sludge in agriculture.

Benthic release of BOD_5 is also demonstrated in the sampling results between the anaerobic ponds influent and effluent: 30 percent increase in the total BOD fraction occurs (due to solids) and over 100 percent increase in the soluble (filtered) portion of the BOD_5.

The increase in the concentrations of Total Kjeldahl Nitrogen (TKN) and ammonia through the anaerobic ponds also supports the established hypothesis that under anaerobic conditions the majority of the organic nitrogen is converted via bacterial decomposition and hydrolysis into ammonia nitrogen.

A microbiological analysis of the ponds was also conducted. The microbiological analysis of the ponds has one important objective: to diagnose the health of the process and determine the most probable causes for performance problems. Figure 3.6-2 illustrates the algae type and count at various points in the process.

Algae are photosynthetic aerobic organisms that grow with simple inorganic compounds (CO2, NH3, NO3-, and PO4--) using light as an energy source. Algae produce oxygen during the daylight hours and consume oxygen at night.

Algae are desirable in facultative lagoons since they generate oxygen needed by bacteria for waste stabilization. Three major groups occur in lagoons, based on their chlorophyll type: brown algae (diatoms), green algae, and red algae. The predominant algal species at any given time is dependent on growth conditions, particularly temperature, organic loading, oxygen status, nutrient availability, and predation pressures.

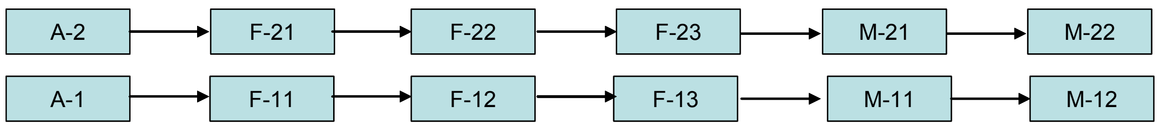
A fourth type of "algae" common in lagoons is the cyano-bacteria or blue-green bacteria. These organisms grow much as the true algae, with the exception that most species can fix atmospheric nitrogen. Blue-green bacteria often bloom in lagoons and some species produce odorous and toxic byproducts.

The absence of algae in the first stages of facultative ponds is a clear indicator of overloaded facultative ponds.

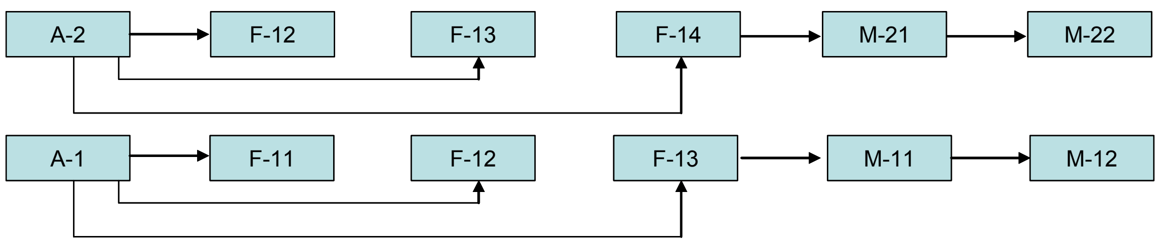
It appears that ponds F-11, F-12, and F-13 have been operating primarily as anaerobic reactors, eliminating the two other essential zones of a "healthy facultative lagoon": the facultative and the aerobic zones.

The low algae counts in the last ponds, M-11, M-21, M-12, and M-22 is also a sign of organic overloading reaching the last treatment units of Mafraq's wastewater treatment plant.

A reduction of the organic loading to Train No. 2 for even a short period of time (approximately 2 months in this case) resulted in significant improvements in the operating conditions in pond F-23, as seen in Figure 3.6-2. Algae count in F-23 was 64,800 count/mL versus 7,110 count/mL in the operating parallel cell, F-13. The following flow sheet summarizes the current configuration, showing the limited flexibility in the initial design.



The following flow sheet provides more operating flexibility, as long as the size of the ponds are adequate and designed according to modern WSP design practices.



3.7 WSP Sludge Characteristics

The metal content measured in the accumulated solids at the bottom of the ponds is very low for the majority of constituents of importance to land application. The majority of the regulated heavy metals was found in concentrations well below the Jordanian Standards JS1145/1996 for use of sludge in agriculture, and should result in acceptable beneficial reuse of the solids.

Table 3.7-1 summarizes the concentration in the ponds, in comparison with the standard JS1145/1996.

Table 3.7-1

Maximum Level for Chemical Elements Concentrations in Sludge (mg/kg Dry Sludge)

Element	Jordanian Standard JS 1145/1996	Concentration in Sludge in the Anaerobic Ponds	Concentration in Sludge in the Facultative Ponds	Concentration in Sludge in the Maturation Ponds
Arsenic	75	N.D.	0.45	N.D.
Cadmium	85	0.12	0.17	0.6
Chromium	3,000	1.84	1.94	3.45
Copper	4,300	7.51	10.61	16.4
Lead	840	1.95	2.34	4.65
Mercury	57	N.D.	N.D.	N.D.
Molybdenum	75	1.38	2.33	5.63
Nickel	420	1.0	1.02	2.0
Selenium	100	N.D.	1.04	N.D.
Zinc	7,500	28.4	35.2	53
Cobalt	150	N.D.	N.D.	N.D.

Sludge concentrations, shown on Figure 3.6-3, suggest that volatile matter is drifting from cell to cell (high VSS concentrations in the FP). The average TSS concentration in the settled sludge at the bottom of the anaerobic ponds is about nine percent, in the facultative pond about eight percent, and close to 10 percent in the maturation ponds. Such high concentrations of solids in the later stages of a WSP system are a sign of "biological solids" overflowing from pond to pond.

The high organic content, in the last treatment units suggest that all ponds are operating in anoxic conditions. The absence of oxygen prevents nitrification from occurring, as seen from the results in Figure 3.6-1. Ammonia and TKN are similar in the influent and effluent composite samples.

As far as pathogen removals, it appears that current operational conditions (overloaded facultative cells) prevent the establishment of a healthy "predator" population in the ponds. Consequently the pathogens count is almost unchanged from pond to pond.

3.8 Major Performance Limiting Factors

Several major performance-limiting factors were identified. They are listed in approximate order of importance below:

1. All cells of the plant are smaller than the sizes cited in several key plant documents. Under the current process flow sheet arrangement, the facultative ponds are undersized by a factor of 25 (available surface area of F-11 and F-21 is 0.71 ha, while the required area, excluding the benthic feedback from A-1 and A-2 must be at least 17 hectares in the summer and 35 in the winter).
2. Facultative ponds operated in series are leading to significant organic overloading, because they cannot be operated in parallel during the colder periods of the year for example.
3. Long gravity sewer from Mafraq to the WWTP functioning as anaerobic reactor.
4. Influent and effluent feed lines to each pond located in the middle of the short sides of the facultative cells, with suspected short-circuiting, while design practice recommend "opposite corners" location.
5. Inefficient flow splitting between the ponds, (organic and hydraulic overloading of Trains 1 and 2).
6. High benthic feedback of organics from the settled sludge in the anaerobic ponds into the supernatant to the first facultative cells.
7. Lack of efficient sludge management facilities; solids dredging/pumping equipment, sludge drying beds.
8. Lack of laboratory testing facilities on the site.
9. Lack of maintenance facilities on the site, leading to equipment deterioration (influent screen, distribution valves, etc.).
10. Poor flow measurement.

The rehabilitation alternatives discussed in the next section provide a means to correct these performance-limiting factors.

Section 4 Rehabilitation Alternatives

This section presents first, a general discussion of the possible upgrading options for an overloaded wastewater treatment plant, then in more details, the rationale and the details of the proposed rehabilitation approach for Mafraq.

4.1 General

The ideal wastewater management strategy for a community is one that requires the least capital, minimizes operation and maintenance costs, complies with the permit requirements, and is reliable.

The wide variety of wastewater treatment options may be broadly categorized as follows:

A. **Higher Technology Options**
   - Mechanical plants
   - Physical – chemical plants

B. **Lower Technology Alternatives**
   - Lower technology, transitional approach (transitional approach identifies processes that are intermediate between the natural purification processes found in nature and a higher degree of mechanization type systems)
   - Lower technology, natural system approach (natural systems describes wastewater systems that are very similar to the naturally occurring purification processes)

The technology selection is strongly affected by the degree of treatment required. Also, the effluent requirements for any specific constituent (organic content, nitrogen content, etc) will dictate what type of process must be selected, designed, built and put into operation.

In addition, non-technical factors should play a significant role in the selection process. Unfortunately, they are very frequently omitted from the initial screening and selection process.

They include:

- Availability of skilled operators, knowledgeable in treatment process fundamentals, including advanced microbiology even for low-technology plants.
- Readily-available spare parts (mechanical plants).
- Administrative procedures supportive of training, communication, etc.
- Cost of operation, mainly electricity and chemicals, including the reliability of the supplies.

In Jordan, a country with very limited water resources, most of the treated wastewater is reused in agriculture. Recently, the Government of Jordan updated the standards for treated wastewaters, and tightened some of the limits believed to have a significant impact on receiving streams. One regulated parameter, the nitrate-nitrogen is expressed in terms of nitrate limits (NO_3), rather than nitrate-nitrogen (NO_3-N). In the 2002 Edition of the Standards (reference 23), the nitrate concentration for discharge into wadis is limited to 45 mg NO_3/L, equivalent to about 10 mg NO_3-N/L. Designing a wastewater treatment facility that consistently meets 10 mg/L of nitrate-nitrogen requires significant stability of the nitrification/denitrification process. Current experience with modern nutrient removal processes suggests that both higher and lower technologies can be used in a Biological Nutrient Removal Facility (BNR). More than one treatment process might be required however for consistent efficiency of the BNR facility, in addition to significant treated effluent recycling to the front end of such plant.

The following brief discussion on the potentially applicable technologies for the upgrading of Mafraq wastewater treatment facility will focus mainly on wastewater treatment systems for nitrogen control, because the conventional pollutants (BOD, TSS, COD, etc) can be removed easily by the selected treatment processes. Meeting nitrate-nitrogen concentrations of 10 mg/l is the main controlling parameter for the design of this plant. Removal of the other standard pollutants by any individual process or a combination of the same processes can be achieved consistently, as proven in numerous facilities throughout the world. On the other hand, nitrification/denitrification processes rely on the proper operation of specific microorganisms, affected by numerous environmental conditions, such as:

- Temperature
- pH
- Alkalinity
- Toxic compounds
- Etc.

Successful wastewater treatment is also dependent on operator understanding, responsible administration, and sound design. Failure of any agent of these (i.e. operator, administrator, or designer) to respond adequately to his/her charge inevitably results in process upset and eventual failure.

The designer can mitigate the failure opportunities at treatment facilities by:

- Selecting robust, flexible treatment processes with conservatively designed, responsive sludge processing and disposal schemes.
- Recommending and supporting operating training.
- Working with the owner to ensure support of the needs of the facility.

In the most recent revised edition of the Manual of Practice, Wastewater Treatment Plant Design, (reference 24), the authors summarized the findings of a recent EPA survey of 150 small plants (capacity less than 4,000 m3/day falls in the category of "small plant") with debilitating problems. The main conclusions of this study are summarized and presented below.

“Conclusion No. 1 – Activated Sludge Process may not be a good design choice for many small plants.

- Give consideration to simpler, more tolerant treatment processes (e.g., fixed media and natural systems) that are less dependent on highly skilled operators.
- Select a treatment technology based on a realistic appraisal of operational and maintenance (O&M) costs (including conservative estimates of sludge quantity and concentration for sludge treatment and disposal, staff salary, recruitment and training, equipment maintenance and replacement, and administrative costs)”.

“Conclusion No. 2 – Plant inflexibility undermines operability

- Designers should conscientiously build flexibility into systems (e.g., piping configurations, redundant unit processes, variable speed pumps, aeration equipment, and equalization tanks for extremes such as infiltration/inflow and return of discontinuous sludge processing streams to the liquid processing train)”.

“Conclusion No. 3 – Small plants have front- and back-end problems with process design.

- Pumps, piping, and aeration systems should be designed to accommodate increased sludge and rags in the system when primary treatment is not provided.
- Operators should be made aware of the need to remove floating debris that passes primary screening.
- Designers should consider finer bar screens, especially when primary sedimentation is not provided and, once screenings and floatable material are removed from the liquid processing train, provide practical facilities to facilitate their permanent removal instead of their internal recycle (and buildup).
- Sludge handling facilities should have the capability to remove and dispose of properly stabilized liquid sludge”.

“Conclusion No. 4 – Heavy loads can confront both skilled and unskilled operators.

- Community administrators and design engineers should frankly discuss and agree on realistic loadings for the facility in the planning process (a conservative, if not skeptical, design approach should be taken to accommodate infiltration/inflow (I/I), industrial loadings, and unusual conditions” (in this case very long sewer line from Mafraq to the WWTP).

“Conclusion No. 5 – Staffing difficulties aggravate poor performance.

Administrators should seek to attract and maintain a high quality staff through increased operator status and visibility using at least one full-time position with a salary comparable to other critical municipal functions (e.g., the police chief) and with reasonable authority for budgeting, purchasing, hiring and firing. Administrators should also provide reasonable opportunities for training and certification”.

“Conclusion No. 6 – Plant budgets may be too low.

- Better fiscal management must start with a separate budget for the treatment plant that includes a sinking fund to cover replacement of major equipment, and that supports adequate staff salaries as well as training and required certification courses”.

“Conclusion No. 7 – Municipal support is a subtle, but vital need.

- Outreach and information transfer must be applied to increase community support; consider making the treatment plant into a multi-use facility that accommodates recreational facilities and shares offices and building space with other community agencies and organizations”.

"Therefore, in municipal wastewater treatment, the designer and owner should lean toward facilities that are low-maintenance, robust and have ample capacity to reflect the uncertainty of staffing, maintenance, and remedial action in a public marketplace where funding of major capital improvements is uncertain and achieved only by public indebtedness with political and public oversight. Both must walk a careful line between providing robust facilities that can respond to a multitude of future uncertainties and an over-design that results in the misuse of public monies for clearly superfluous facilities. When in doubt, trust experience, which strongly suggests that simplicity and harmony with naturally occurring reactions are likely to serve better than a multitude of unit operations for an optimized desktop objective and the temptations of an unproven form of high technology."

Conscious of the drawbacks of the higher-type technologies, and other non-process selection factors in Jordan, we have opted to focus on lower-type technology, maximizing the use of all existing facilities, taking into account the climate, while still providing a rehabilitated wastewater treatment facility in full compliance with the most recent Jordanian Standard, JS893/2002 for discharge to wadis.

4.2 Higher Technology Alternatives

Table 4.2-1 summarizes the status of the nitrogen control alternatives in municipal wastewater applications after some 20 years of experience in the U.S. It covers both higher and lower technology processes. The table is adapted from the most recent edition (2000) of EPA's Manual on Nitrogen Control. The meanings of the two main abbreviations used in the table are: "O" for process in full operation, and "R" for process in some degree of research status. The information provided in this table is usually used as the first "go-no-go" decision taken by the designers, when selecting the most appropriate processes. The discussion is centered mainly on nitrogen removal processes, because the available higher type technology alternatives are proven systems for conventional pollutants in plants similar to the Jordanian conditions.

Table 4.2-1

Status of Nitrogen Control Technologies*

Technology	High			Low
	Well Demonstrated	Limited Application	Found Lacking	Emerging
Higher Technology, Mechanical Plant Approach				
Suspended Growth				
Single Sludge				
Multiphase				
Aerator and/or Aeration Basin Cycling	O,R			
Sequential Batch Reactor	O			R
Multistage (e.g., serial application of processes)	O,R			
Multizone (e.g., ditches)	O,R			
Two Sludge	O,R			
Three Sludge	O,R			
Attached Growth, Single- or Staged Applications				
Submerged Media				
Fluidized Bed		O,R		
Packed Bed				
Down flow	R	R		O
Up flow		O,R		
Non-submerged Media				
Stationary (e.g., trickling filter)	O	O		R
Rotating Media in Solids	O	O		R
Combination Processes				
Any of the Above in Serial Application	O,R	O,R		O,R
Submerged Stationary Media (Vertical Plates or Media)				O,R
Non-submerged				
Stationary Media with Solids Recycle			O	O,R
Rotating Media in Solids Suspension				O,R
Specific Surface Additives to Suspended Growth				
Concurrent Additive Management		O		O,R
Separate Additive Separation & Processing				O,R
Lower Technology, Transitional Natural Systems				
Transitional				
Aerated Lagoons (suspended growth)	O	O		R
Intermittent and/or Recirculating Sand Filtration	O	O,R		O,R
Aquatic-Based				
Lagoons (suspended growth)		O,R		O,R
Facultative (N stripping)				R
Algae Harvesting		R		
Natural and Constructed Wetlands				

Source: Reference 17

O = nitrogen oxidation;

R = nitrogen removal by biological denitrification

4.3 Lower Technology Alternatives

Lower technology approaches are not new, nor are they rare. Even today more than 50 million U.S. residences (25 percent of all single-family dwellings) are unsewered, mostly served by septic tanks and/or soil absorption systems for their wastes. Stabilization ponds (or lagoons) in the U.S. number well over 5,000. Specifically designed land treatment systems number over 1,000. Constructed wetlands number more than 500, along with a few aquatic plant systems. Nor are land-based systems always small – Orlando, Florida, for example, uses a rapid infiltration land treatment system (ground-water recharge) with 2,200 L/s design capacity. Duncan Mara reported in his recent Design Manual that more than a thousand facilities are used in different situations throughout several Mediterranean Countries, providing full compliance with all existing regulations. It is generally accepted that lower technologies are more appropriate in countries with lower availability of skilled operations and maintenance personnel, difficulties obtaining spare parts for mechanical plants, and generally lacking reliable financing tools.

Among the variety of technologies used by wastewater professionals, the following lower technology alternatives are discussed in some detail, because they will form the basis for the rehabilitated wastewater treatment facility in Mafraq.

Stabilization Ponds or Waste Stabilization Ponds (WSP)

Stabilization ponds (lagoons) may have many forms, but the facultative lagoon is the most widely used. Facultative lagoons are large in size, perform best when segmented into at least three cells, obtain necessary oxygen for treatment with biological degradation, and produce large quantities of algae, which limit the utility of their effluent without further treatment. Therefore, simple solids retention/removal supplemental systems, such as sand or rock filters, are often used to upgrade a WSP and meet more stringent organic, solids and nutrient standards.

Aerated Lagoons (AL)

Aerated lagoons use mechanical equipment to enhance and intensify the biodegradation rate. They do not produce the intense algal load on downstream processes and have smaller area requirements than facultative systems. Aerated lagoons are sometimes called “aerated stabilization basins” (ASBs), a term used to describe thousands of operational facilities serving the pulp and paper industry throughout the world. In aerated lagoons, oxygen is supplied mainly through surface aerators or diffused aeration (fixed or floating), rather than by algal photosynthesis.

Constructed Wetlands Systems (CW)

Constructed wetlands are large basins filled with wastewater undergoing some combination of physical, chemical and/or biological treatment processes that render the wastewater more acceptable for discharge to the environment. These systems perform best when divided into a minimum of three zones, the first and last being fully vegetated with macrophytes (cattails or bulrushes) and the middle having an open water surface, which performs like a facultative lagoon. In the first zone, the suspended solids are flocculated and settled under anaerobic conditions. The second zone re-aerates the anaerobic wastewater to provide oxygen for aerobic degradation and possible nitrification before final-zone flocculation, sedimentation and denitrification steps. For Mafraq, a simplified, type of constructed wetlands will be used: the reed bed process.

Recirculating Sand Filters (RSF)

Recirculating sand filters (RSF) are a modified version of the old-fashioned, single-pass open sand filter. They were designed to alleviate the odor problems associated with sand filters. Odors are minimized through recirculation, which increases the oxygen content in the effluent that is distributed on the filter bed. RSF remove contaminants in wastewater through physical, chemical, and biological processes. The Recirculating Sand Filter process is similar to the Intermittent Sand Filters process, and the terms are often used interchangeably. We will refer to this technology as "RSF" in this report, for consistency with other similar projects currently underway in Jordan.

Intermittent Sand Filters (ISF)

The intermittent sand filters (ISF) are single-pass open sand filters, dosed periodically from a large dosing basin. They are sized based on hydraulic loading, and must be operated with dosing and resting periods for full treatment capacity recovery. ISFs remove contaminants in wastewater through physical, chemical, and biological processes. They are used in several small and large U.S. communities, providing low cost nitrification, as well as sludge and organics treatment of the effluent from other treatment units.

Advanced Integrated Pond Systems (AIPS)

The Advanced Integrated Pond System (AIPS) is a relatively new wastewater treatment process (a modification of the WSP process). An AIPS system involves the addition of a digestion pit at the bottom of the first pond of a more conventional WSP system. This first pond in an AIPS system is called Facultative Pond, because the surface area is aerobic, the middle facultative, and the "digestion pit" anaerobic. The second pond in an AIPS system, called "high rate algal pond" is a shallow reactor providing ideal conditions for intense algal growth. The third pond, called algal settling pond collects the produced algal biomass, which needs either manual or mechanical removal. Recently several AIPS systems have been upgraded with dissolved air flotation (DAF) systems in municipalities with stringent effluent limitations. The system was developed by Dr. Oswald at the University of California at Berkeley (UCB).

The majority of the existing AIPS facilities are designed for low-strength municipal wastewaters (BOD/TSS concentrations lower than 300 mg/L). There are few facilities handling high BOD industrial wastes, and none (known) treating high TSS concentrations. Excessive accumulation of sludge in an AIPS system would negate the claimed benefits of "zero sludge production," and could change dramatically the operational conditions of a wastewater treatment plant.

Although there are a limited number of installations, the AIPS was investigated to determine if the process was suitable for treating high TSS wastewater. The influent wastewater composition and performance data for several AIPS systems were examined and compared to the typical waste characteristics of the influent to the Mafraq WWTP. In Mafraq for example, close to 2,000,000 kilograms of TSS per year will be generated in the near future. At approximately 4 percent solids content, this mass represents a volume of about 45,000 cubic meters. In our opinion, such mass cannot be stored in a "small sludge digestion pit" in the first pond. Using established design standards, the digestion pit alone would occupy upwards of 43 ha. Therefore, an AIPS system does not appear to be an either practical or feasible for this project. .Some type of simple, compact, non-mechanical solids separation equipment would be the most appropriate solution to the problem of high TSS content.

As shown in Table 4.3-1, there is a significant difference in influent wastewater composition between the existing US-based AIPS facilities and the influent wastewater characteristics for Mafraq. The AIPS is not recommended for an upgrade similar to that of Mafraq's WWTP, due to the higher influent wastewater strength, limited number of installations, and rather strict nitrate-nitrogen effluent standards.

Table 4.3-1

Comparison of Existing AIPS Wastewater Characteristics with Mafraq

Parameter	Units	St-Helena AIPS*	Richmond AIPS*	Influent Mafraq
BOD	mg/L	223	236	232 to 2,814
COD	mg/L	438	Not given	436 to 5,218
TSS	mg/L	Not given	202	164 to 2,947
TVS	mg/L	604	182	130 to 2,350
TKN	mg/L	40	45	100 to 200
Ammonia	mg/L	Not given	37	100 to 184

* Source: Water Science and Technology Journal, Low-cost and energy-saving wastewater treatment technologies Volume 24, Number 5, p.1-9

Waste Stabilization Ponds (WSP), Aerated Lagoons (AL), Constructed Wetlands (CW), and Intermittent Sand Filters (ISF), as well as the Recirculating Sand Filters (RSF) are often referred to as low-rate systems, which require little or no mechanical equipment. Often a combination of several low-rate systems is required to meet stringent effluent standards. When properly designed and operated, low-rate systems can produce a final effluent comparable to high-rate systems, which use mechanical equipment.

Table 4.3-2 summarizes the advantages and disadvantages of both the higher and lower technologies using simple screening criteria: the land required for the process and the ability of the technologies to meet nitrogen control objectives.

As it can be seen from Table 4.3-2, a combination of more than one unit treatment process, is required in Mafraq, in order to meet the most stringent effluent criteria: nitrate-nitrogen of 10 mg/l, required for effluents discharged into wadis.

As indicated above, a combination of low-tech processes is required to reduce the projected influent and parameters below the JS 893 2002 standards of:

BOD < 60 mg/L
TSS < 60 mg/L
COD < 150 mg/L
Nitrate-Nitrogen < 10 mg/L
FC < 1,000 MPN/100 mL

The rationale for combining the unit processes described below is based on the simplicity of each system, their low capital cost, robustness, ability to handle excursions from the average design values and low operational and maintenance costs. Each treatment unit, retained for the rehabilitation of Mafraq WWTP was selected, because it will fulfill the following treatment function:

- The Sedimentation/Thickening Tanks (SDT) will reduce the solids content of the wastewater
- The Sludge Drying Beds (SDB) will reduce the water content of the sludge
- The Denitrification Reactors (PDN) will reduce the nitrates in the recycled stream
- The Aerated Stabilization Basins (ASB) will reduce the BOD content and provide suitable environment for some nitrification
- The Facultative Lagoons (FL) will reduce the biosolids content of the treated wastewater
- The Recirculating Sand Filters (RSF) will provide additional nitrification/ denitrification, as well as solids removal
- The Constructed Wetlands (CW) will provide residual BOD removal, as well as additional nitrification/denitrification
- The Maturation Ponds (MP) will provide additional pathogen removal
- The chemical disinfection step (DB) will ensure full destruction of the regulated pathogens

Table 4.3-2

Screening Criteria for Nitrogen Control Systems*

HIGHER TECHNOLOGY, MECHANICAL PLANT APPROACH									
		<1	1.0+/-	Easy	Easy	Easy	Easy	Easy	Possible
LOWER TECHNOLOGY, TRANSITIONAL APPROACH									
Aerated Lagoons	-	1 - 12	3 -100	Possible	Possible	Doubtful	Possible	Possible	Doubtful
Intermittent Sand Filters	-	<1 - 4	-	Easy	Possible	Doubtful	Doubtful	Doubtful	Doubtful
Recirculating Sand Filtration with Anoxic Rock Filter Enhancement	-	3 - 4	1-2	Easy	Possible	Doubtful	Easy	Possible	Doubtful
LOWER TECHNOLOGY, NATURAL SYSTEMS APPROACH									
Aquatic Based Technologies									
Facultative	-	3 - 8	20-100	Easy	Possible	Possible	Possible	Possible	Doubtful
Algal Harvesting	-	4 +	10+	Easy	Easy	Possible	Easy	Possible	Doubtful
Constructed Wetlands									
Free Water Surface	-	10 - 12	7-10	Possible	Possible	Doubtful	Possible	Possible	Doubtful
Submerged Flow	-	2 - 4	1-2	Doubtful	Doubtful	Doubtful	Doubtful	Doubtful	Doubtful
Land-Based Technologies									
Slow Rate (Moderate)	-	20 - 280	-	Easy	Easy	Possible	Easy	Easy	Possible
Rapid Infiltration (High)	-	1 - 24	-	Easy	Possible	Doubtful	Possible	Possible	Doubtful
Overland Flow (Low)	-	6 - 44	-	Easy	Possible	Doubtful	Possible	Doubtful	Doubtful
Subsurface Infiltration (Mod-High)	-	3 - 32	-	Easy	Possible	Possible	Possible	Possible	Doubtful

*Source: Reference 17

4.4 Recommended Approach to Mafraq WWTP Rehabilitation

These recommendations should provide a cost-effective solution for upgrading the existing wastewater treatment facility to comply with Jordanian Standard JS 893 2002. The technologies selected are proven, reliable wastewater treatment processes that require minimal operator intervention.

4.4.1 Design Criteria and Treatment Objectives

The rehabilitated wastewater treatment facility, serving the future (2025) sewered population of the Town of Mafraq will be designed based on the following design criteria:

- Average flow of 6,550 m^3/day.
- BOD_5 load of 4,740 kg/day.
- TSS load of 4,740 kg/day.
- TKN load of 800 kg/day.

The plant effluent will meet the most recent Jordanian standards JS 893 2002 for discharge to wadis.

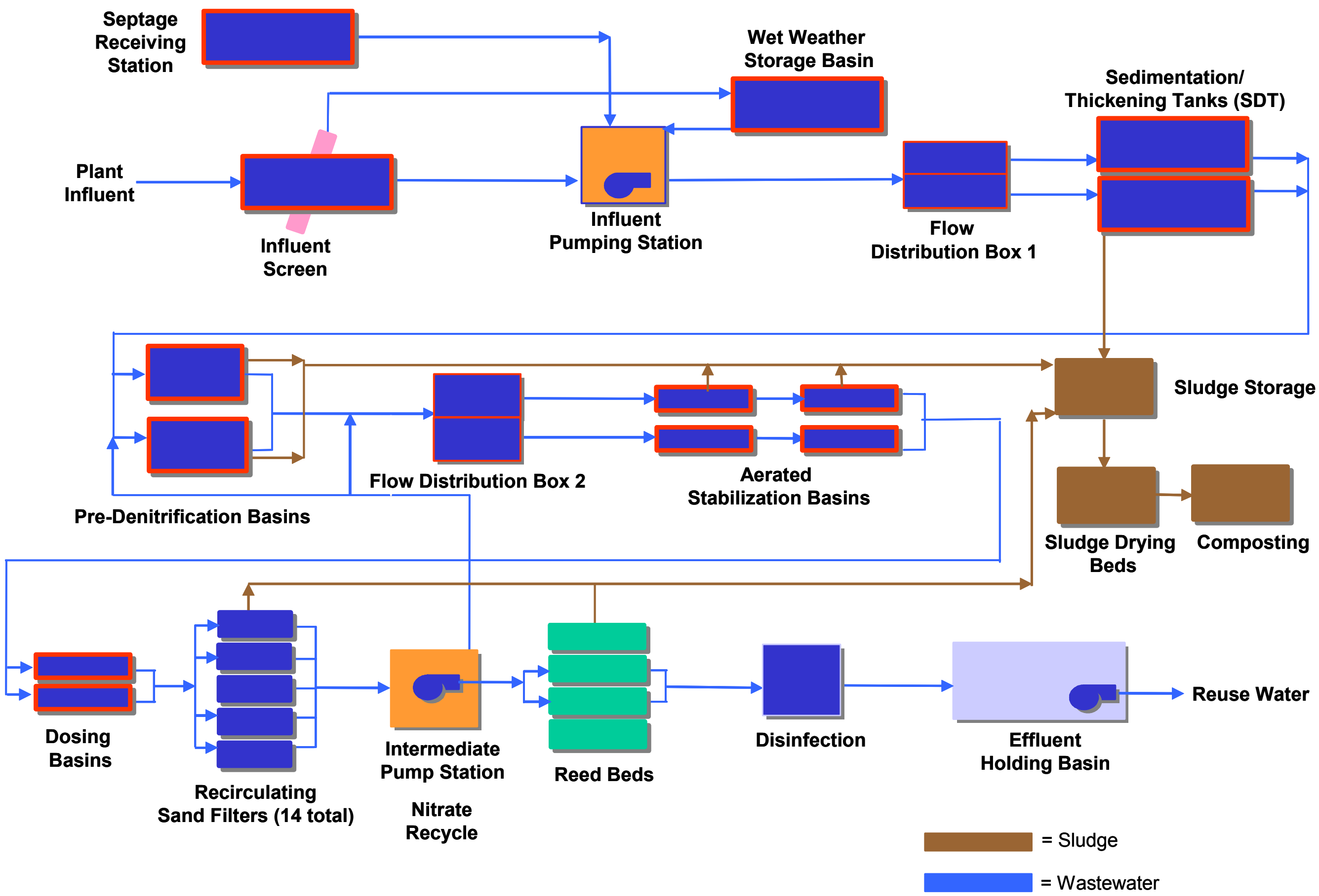
4.4.2 Process Flowsheet

Figure 4-4.1, presents the process flowsheet for Mafraq rehabilitated/upgraded facility. Designed with influent loads and flows summarized previously, the plant will offer sufficient flexibility to handle with one, two, or more treatment steps the current, the intermediate (2010, 2015) conditions, and finally, with all units in operation, the ultimate (2025) situation. Figure 4.4-2 shows a simplified process and mass diagram, showing the expected organic and flow concentrations around each treatment unit.

Insert Figure 4.4-1

Figure 4.4-2

Process and Schematic Flow Diagram



The high level of suspended solids in the influent to the Mafraq WWTP was considered to be an important design parameter that led to the decision to abandon the long retention time anaerobic ponds at the head of the plant. The long influent pipe between the City and the plant actually fulfills the same function. In addition, significantly higher TSS loads might be flushed during high storm/rain events. It was thus considered to be more appropriate to introduce sedimentation/thickening tanks upstream in lieu of the anaerobic lagoons, and thus prevent those high TSS loads to enter, settle, and fill the subsequent treatment units. This will eliminate the concern for benthic release and the associated BOD increase that would continue to occur with anaerobic ponds at the head of the plant.

The proposed flowsheet integrates several of the existing structures, most of which were found to be in good condition during our visit to the facility in May 2005. The majority of the ponds are included in the proposed flowsheet. The volume of the facultative ponds, converted to aerated lagoons, can be increased substantially by demolishing the dividing wall between each pair of two parallel facultative ponds (the wall between F 11 and F 21, as well as F 12 and F 22). Other treatment units can be kept intact, or modified in similar fashion, according to decisions made during the detailed design steps. This approach to augmenting the process capacity provides an opportunity to raise the berms and also add storage capacity for periods of high flows.

Using a staged construction sequence will maintain the existing wastewater treatment plant operational during the upgrading process. The following construction sequence will ensure continuous operation of the facilities:

- Construction of all units in red on Figure 4.4-1 (Phase 1). The added treatment units (headworks, lagoons, filters, etc.) are located besides the existing WSP.
- Modification of existing units, represented in green on Figure 4.4-1 (Phase 2). For example each pond that must be modified will be taken out of service, modified, put back in operation, before the next pond is taken out of service for rehabilitation.
- Total system integration, optimization and commissioning (Phase 3).

4.5 Description of Unit Processes

We recommend a liquid process treatment train consisting of very efficient influent screening, a wet weather storage lagoon, influent pumping station, sedimentation/thickening tanks, predenitrification reactors, aerated lagoons, recirculating sand filters, constructed wetlands and treated effluent storage. The sludge treatment train will include an aerated sludge stabilization/storage lagoon, sludge drying beds for dewatering. Dewatered sludge from the sludge drying beds will be stabilized further in windrow composting cells, then trucked to a landfill for final disposal or reused beneficially by nearby farmers. Figure 4.5-1 shows the individual treatment processes with the illustrative photographs. The proposed facility will have two treatment trains, with possibility to divert from one train to another for improved operational flexibility. A brief description follows.

New Headworks Facilities (SSR)

Wastewater conveyed to the treatment facility will initially undergo influent screening before treatment. The screenings facility will consists of a mechanical screen and a manual bar rack. The mechanical screen and manual bar screen will be located in separate channels with removable aluminum grating. Each channel will be equipped with slide gates to allow a channel to be isolated for maintenance. Screenings removed from the wastewater will be discharged into an open dumpster located downstream of the mechanical screen.

In effort to minimize the size of the downstream treatment units, the screenings channel will be equipped with an emergency overflow weir. The emergency overflow (after screening by a manually cleaned screen) will be set at an elevation to divert to wet weather storage lagoon flows that exceed approximately 13,000 m^3/day. A gravity sewer with a slide gate located at the Headworks Facility will be provided to allow wastewater diverted to the wet weather storage lagoon to be returned to the screenings facility following the wet weather event.

Insert Figure 4.5-1

The mechanical screen recommended for the Mafraq WWTP uses shaftless spiral technology to perform screening, sludge conveying, and dewatering in one step. This type of screen will minimize odors and the volume of screenings requiring final disposal.

The existing headwork facilities will be used during the interim period of upgrading. Once the new units are operational, the old one could be disconnected and decommissioned. They would allow using the existing anaerobic ponds until the first aerated lagoon is put in operation.

New Septage Receiving Station (SRS)

The septage receiving station will allow separation of solids and liquid from a predetermined number of tanker trucks, down loading waste from the rural areas of Mafraq. The exact volumes and the associated load of pollutants will be determined in consultation with WAJ.

New Influent Pump Station (PSP)

Following influent screening the wastewater will be directed to an influent pump station equipped with three submersible wastewater pumps (2 duty and 1 standby). Each pump will be rated for a capacity of 6,500 m^3/day. The pumps will lift the wastewater into a distribution box. The screened influent will be fed to the next unit, the distribution box, and be divided equally between the two sedimentation/thickening tanks.

New Distribution Box

The distribution box will split the flow evenly between two sedimentation/thickening tanks. The distribution box will be equipped with stop plates to provide the flexibility of isolating tanks.

New Wet Weather Storage Basin (WWS)

The new wet weather storage basin will collect all flows exceeding 13,000 m^3. These flows will be returned back to the head part of the plant during lower flow conditions for processing and treatment. The basin could share a common wall with the earthen treated effluent storage basin, or be constructed in the vicinity of the headwork facilities.

New Sedimentation/Thickening Tanks (SDT)

The sedimentation/thickening tanks are above-grade, painted carbon steel tanks. The tanks are equipped with plate settlers to maximize solids removal efficiency. The units have no moving parts. The conical bottoms produce a high underflow solids concentration, thereby minimizing the sludge volume. The underflow pipe will be accessible and equipped with a manual isolation valve. Solids removed from the units will be conveyed by the hydrostatic pressure provided in the tank to an aerated sludge stabilization/sludge holding lagoon, then discharged to the sludge drying beds.

New Predenitrification Basins (PDN)

The predenitrification basins will be glass-fused-to-steel round tanks, fitted with bottom sludge removal piping and mixers for improved sludge/effluent contact. To ensure stable denitrification we are adding fixed type media in each tank. Effluent from the RSFs will be recycled to these units, as well as to some other unit processes. Biomass in the unit is expected to fully denitrify the recycled mass of nitrates from the final stages of the treatment process.

New Aerated Stabilization Basins (ASB)

The biological treatment system will include three ASBs in series per train (two trains are suggested for this facility) for carbonaceous BOD removal and nitrification.

ASB 11 would be a new structure, while ASB 21, ASB 12, and ASB 22 will be formed by the existing facultative ponds. Aeration will be provided by floating surface aerators. In the proposed system the first stage ASBs will be complete mix units, followed by partial mix and settling cells. The back end of the settling cells will be equipped with rock filters for enhanced solids removal. Piping and valve provisions will be provided to allow a basin to be bypassed. Although minimal sludge accumulation is anticipated for the aerated lagoons, sludge removal might be required every seven to ten years. Sludge removal either by floating pumps or dredge systems are considered the best choice at this point.

One of the first ASBs (ASB 21) will be formed by removing the intermediate berm between the existing anaerobic ponds A-1 and A-2. The second train ASB is a new unit, built beside the existing facility. The intermediate berm between F-11 and F-21, F-12 and F-22 will also be removed. Such low cost improvements would increase the bottom footprint of each ASB, gain more water volume for wet weather flow storage, and improve the conditions for adequate aeration design of the basin. During the design phase of the project we will evaluate the cost effectiveness of cell formation by floating baffles, instead of individual cells.

New Dosing Basins (DB)

Two dosing basins (earthen structures) are proposed to temporarily store treated effluent from the aerated lagoons. Each dosing basin will be equipped with a motor actuated slide telescoping valve, which will be periodically opened to distribute stored effluent between a series of recirculating sand filters. Although the lagoon effluent is anticipated to contain a low suspended sludge concentration, each dosing basin will be equipped with a mud valve to allow the flexibility to drain the contents to the influent pumping station.

New Recirculating Sand Filters (RSFs)

The RSFs are provided to remove E. Coli, intestinal helminthes eggs, and TSS present in the wastewater to ensure compliance with Jordanian regulations JS 893 2002. Our experience indicates the RSFs will also provide some degree of additional nitrification and denitrification, as well as residual carbon oxidation. Each RSF will include an underdrain system to collect filtrate from the RSFs. The filtrate will be directed to an effluent recycling pump station. A portion of the filtrate containing nitrate-nitrogen will be pumped to the predenitrification basins, as well as to the headworks and/or the constructed wetlands, where it will be mixed with the carbon rich influent and undergo denitrification. The wet well of the internal recirculating pump station will include a series of weirs with removable stop plates to control the volume of wastewater returned to each denitrification system (i.e., recirculation ratio).

New Recirculating Pump Station (RRPS)

Effluent from the recirculating sand filters will be recycled to the front end of the plant. The recirculating line, installed between the two treatment trains, will serve to divert portion of the treated effluent to any treatment unit. Recirculation of disinfected effluent (by chlorine) is not recommended because such stream could be harmful to the biology found in each treatment process.

New Constructed Wetlands (CW)

Filtrate not returned to the sedimentation/thickening tanks will be conveyed to a couple of constructed wetlands (i.e. Reed Beds) for effluent polishing. Several reed beds are recommended for this application to allow the flexibility to take a reed bed out of service for maintenance. The constructed wetlands include subsurface flow within a gravel medium. Process removal mechanisms include biological conversion of any remaining substrate, nutrient removal through plant uptake, filtration, and sedimentation. The upper portion of the bed provides an aerobic environment, as a result of oxygen supplied by the vegetation's root system. Therefore, some additional nitrification will occur in the upper layer. Most of the reed bed however is anaerobic. Therefore, any nitrates produced in the upper layer from the nitrification process or found in the effluent from the RSFs will be consumed by heterotrophic bacteria. The carbon source for denitrification is provided by the influent soluble substrate and decay of the root system.

Water Reuse Storage Pond (WRSP)

Treated effluent from the constructed wetlands will be conveyed to a treated effluent storage lagoon. The storage lagoon will be used to temporarily store treated effluent prior to agricultural reuse. A geomembrane liner may be required, depending on the soil conditions, to minimize seepage. The existing irrigation pumps will be used to withdraw treated effluent from the storage lagoon and convey the water to the disposal fields. If required, pumps with larger capacity might be included in the final design of the system.

The volume of the storage pond has been selected in the 2001 Conceptual Design Report and approved by the water Authority of Jordan. One 90,000m^3 pond will be used to store the excess effluent, not used by the nearby farmers. USAID suggested keeping the same capacity at this stage of the project. The capital cost for this structure is however excluded from this evaluation.

New Sludge Stabilization/Storage Lagoon (SL)

Sludge removed from the sedimentation/thickening tanks will be conveyed to an aerated sludge stabilization/storage lagoon. The aerated sludge lagoon will provide sludge equalization capacity, some degree of degradation of the biological solids to prevent odor emissions, as well as improvement to the sludge dewatering characteristics. Mixing and partial aeration will be provided by floating surface aerators. Periodically the floating surface aerators will be turned off. Supernatant from the aerated sludge holding tank will be decanted and directed to the influent pump station through a gravity sewer. Settled sludge will be pumped to a series of sludge drying beds. The exact type of construction material will be determined during the detailed design process.

New Sludge Drying Beds (SDB)

Sand sludge drying beds are one of the simplest and least expensive processes for dewatering sludge. Dewatering is accomplished by drainage through the sand and evaporation. The sludge drying beds will be equipped with an underdrain system to collect water that has percolated through the sand. A common gravity sewer will collect the water from the underdrain system and direct it to the influent screw pump station.
The drying time depends on the climate, but is anticipated to take 30 to 40 days.

New Sludge Composting Cells (SCC)

The dried sludge will be removed from the sludge drying beds, piled in the composting cells, amended with shredded organic material and stabilized by the windrow composting method. Amendment material from the vicinity of the plant could be used as carbon source, required for the stabilization of the dried solids from the plant.

Existing Disinfection Channels (DB)

Plant effluent disinfection will be achieved in three steps:

Step 1: Using the two facultative ponds
Step 2: Using the four maturation ponds in series or in parallel.
Step 2: Using the existing/modified chlorination facilities.

Additional pathogen dye-off would occur in each treatment process:

- The aerated lagoons.
- The recirculating sand filters.

- The constructed wetlands.
- The effluent storage basin.

Water Reuse Facilities

Figure 4.5-2 shows the overall plant site, including all dedicated water reuse parcels. The entire site is owned by WAJ. The treatment units for the upgraded plant require more land than the existing plant. However, the new treatment units are not occupying any space reserved previously for agricultural water reuse. Water reuse at this location is an essential component of the sound future management of the wastewater, including environmental protection. The final size and configuration of all process units will be developed during the design phases of the project.

These recommendations should provide a cost-effective solution for upgrading the existing wastewater treatment facility to comply with Jordanian Standard 893/2002. The technologies selected are proven, reliable wastewater treatment processes that require minimal operator intervention.

Insert Figure 4.5-2

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